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NOVEMBER, 1950. Vol. XLV, No. 11

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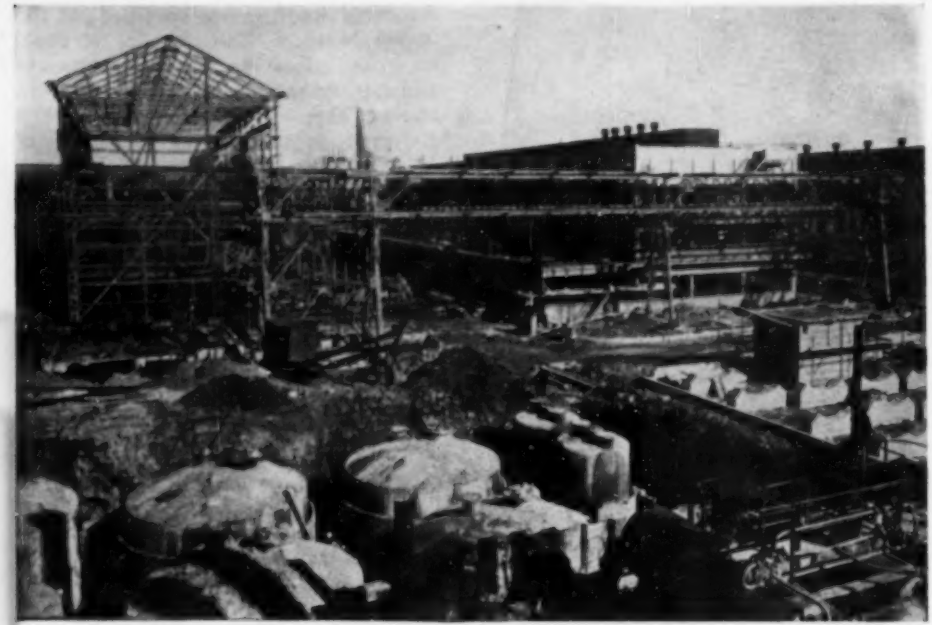

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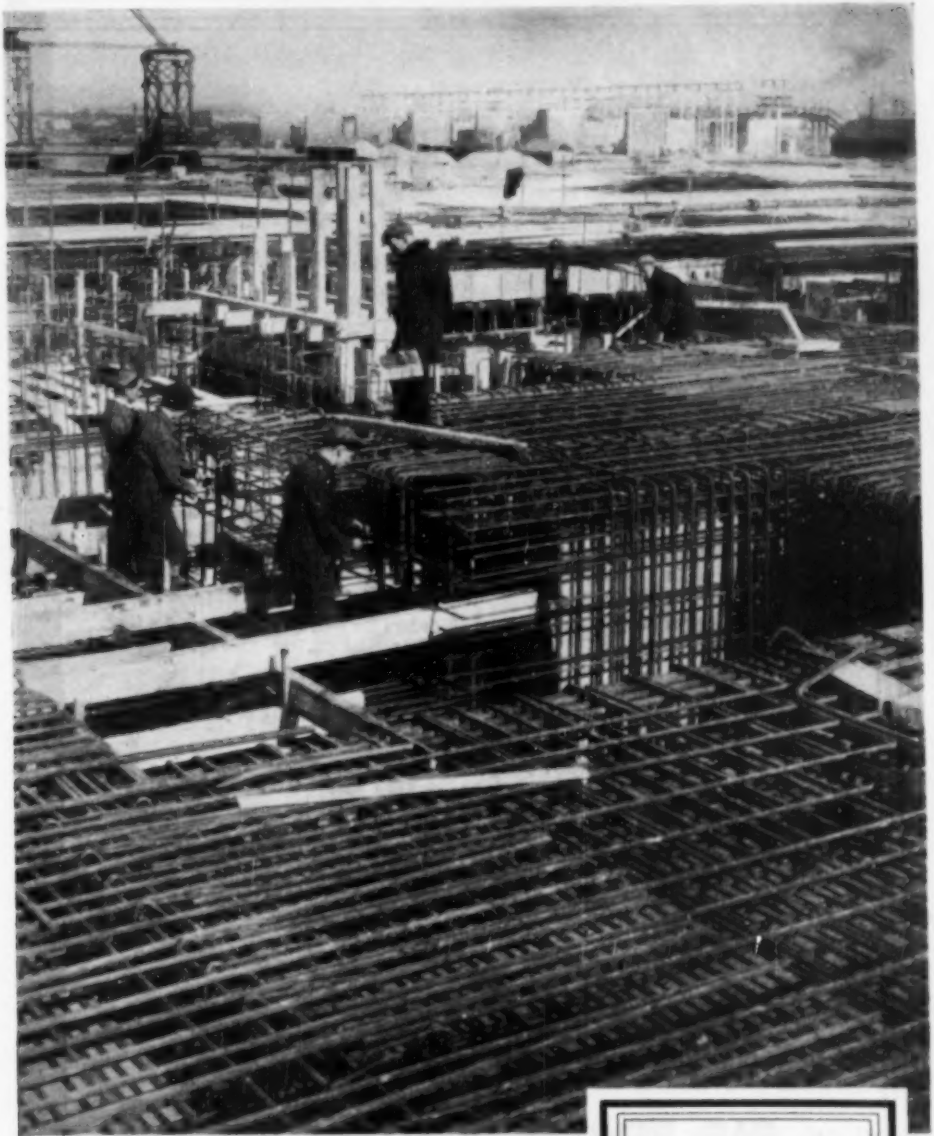

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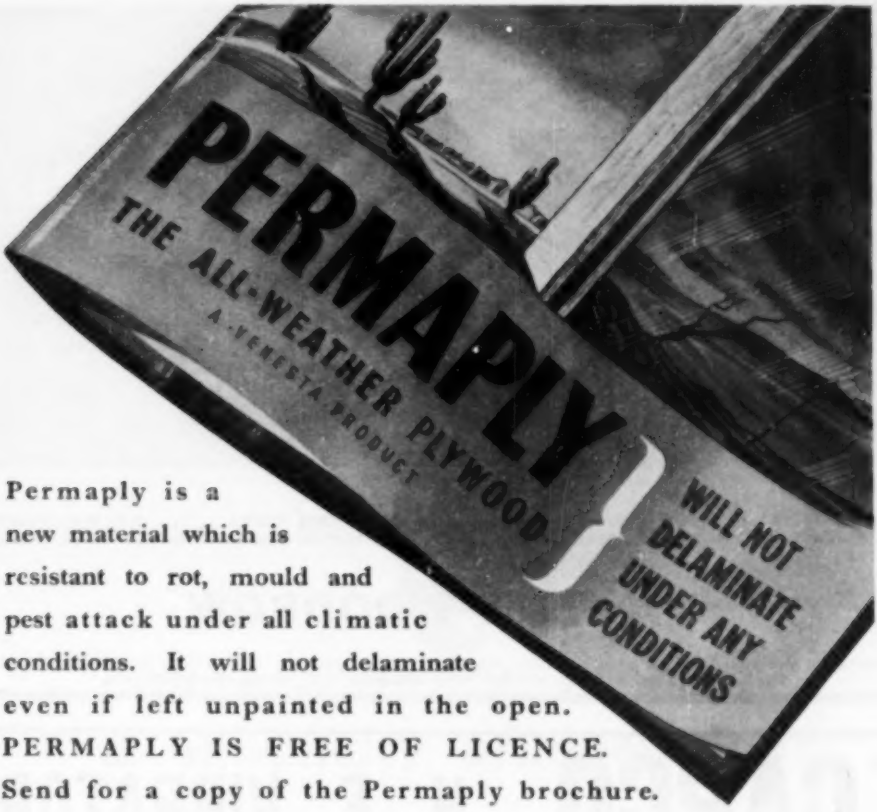

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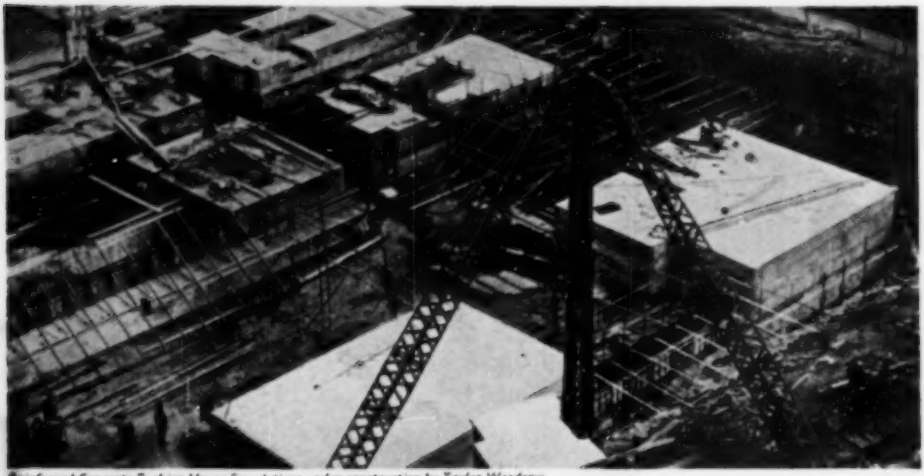

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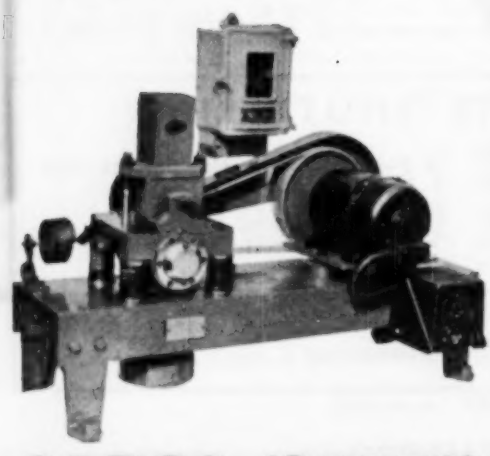

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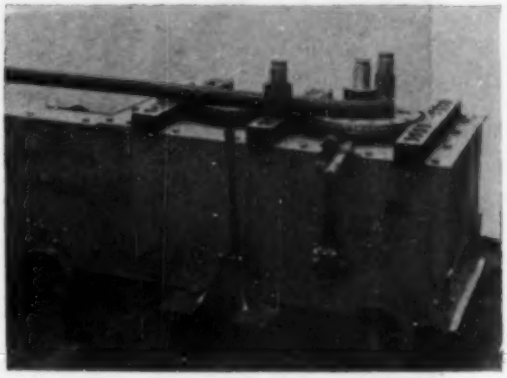

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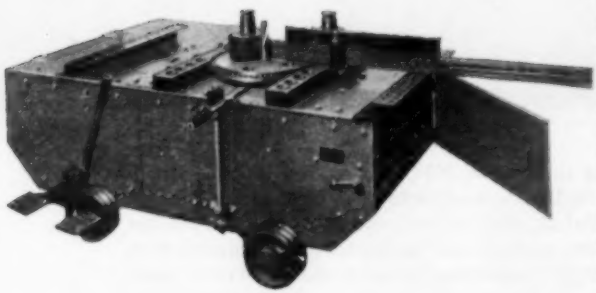

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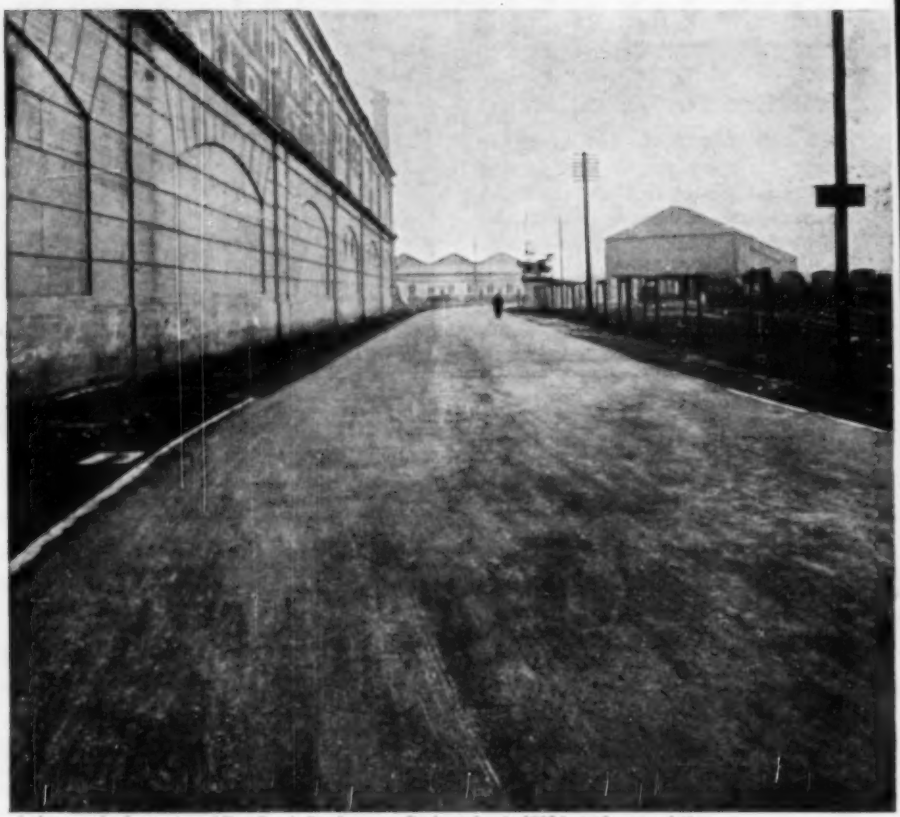

A photograph of a section of Test Road, Southampton Docks, taken in 1917 just after completion.

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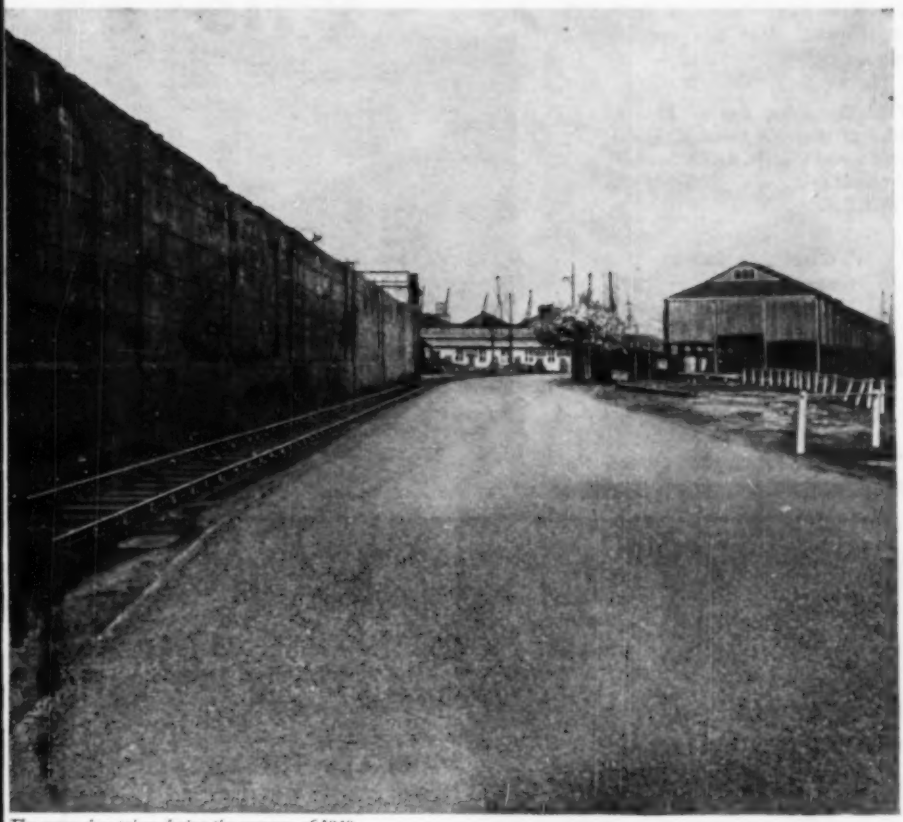

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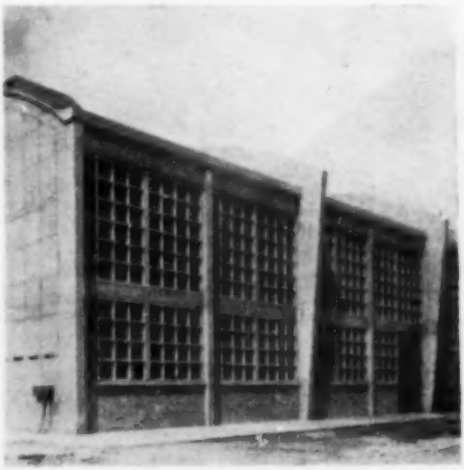

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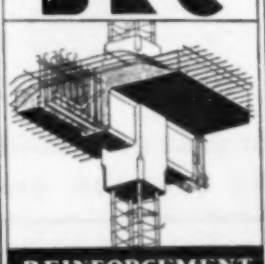

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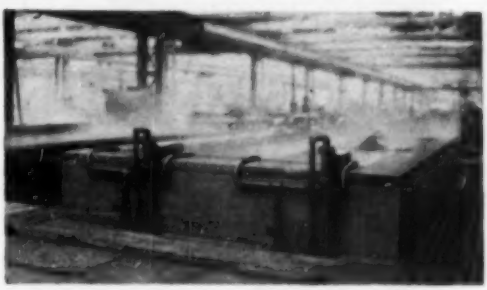

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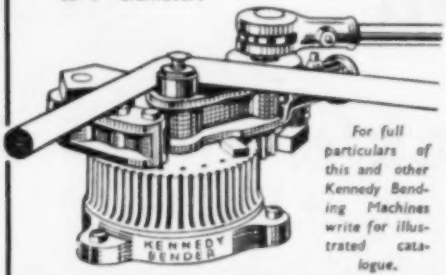

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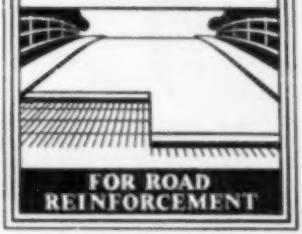

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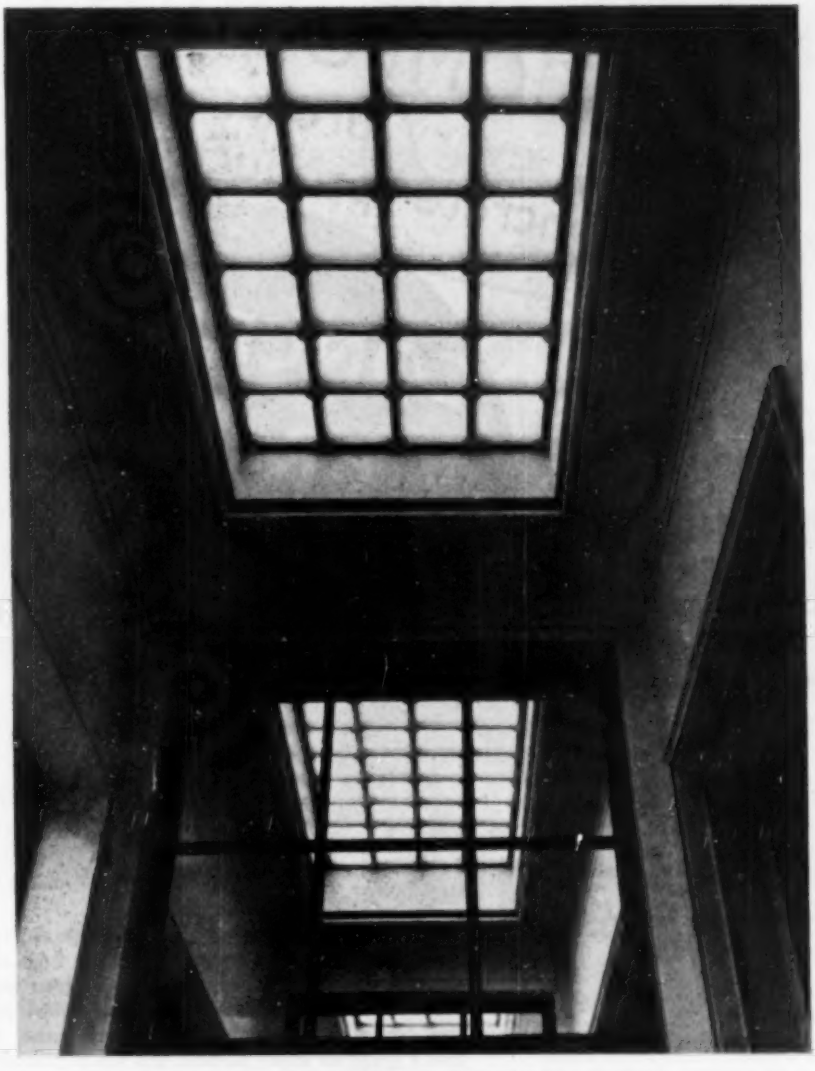

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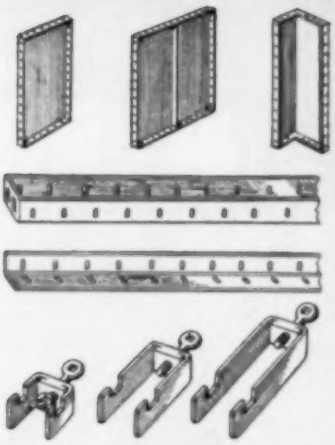

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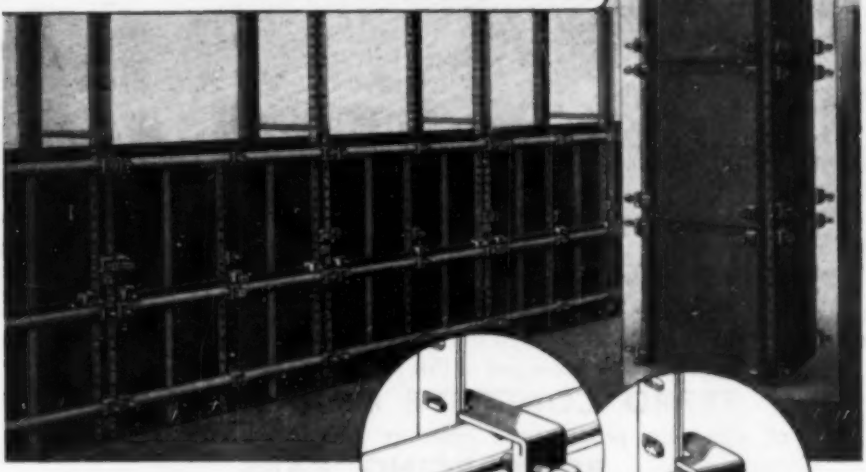

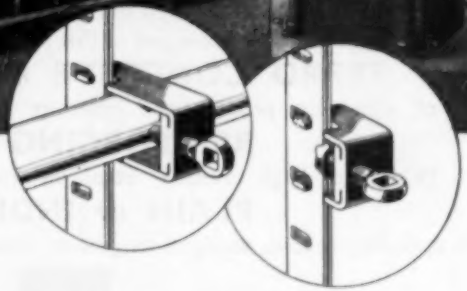

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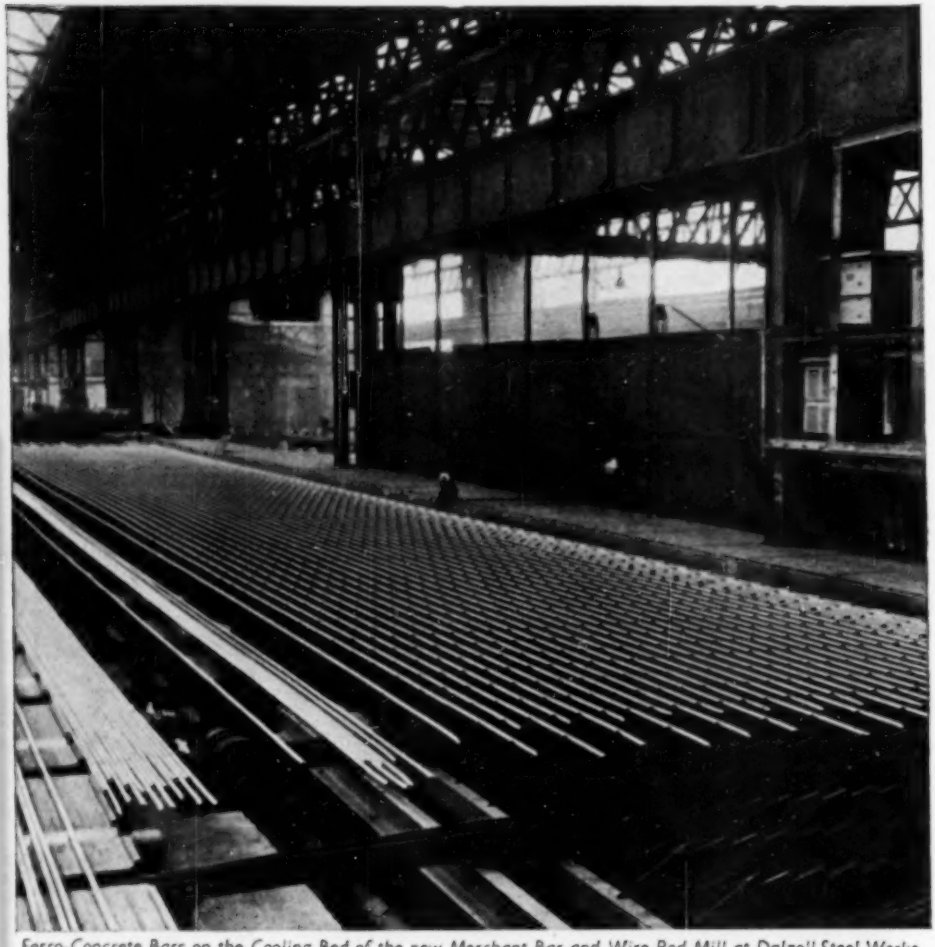

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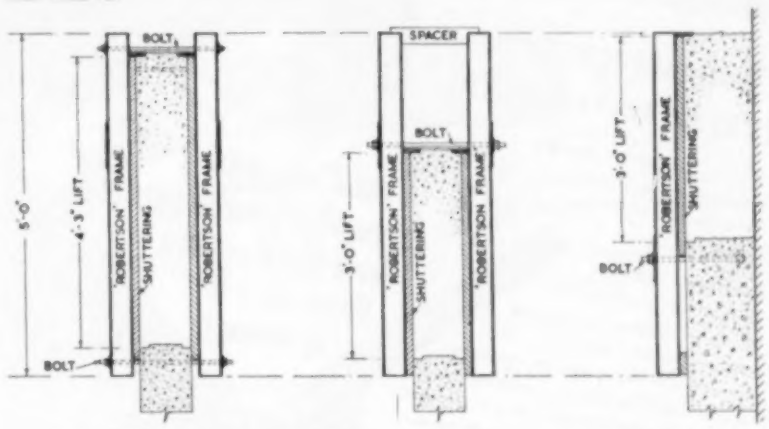

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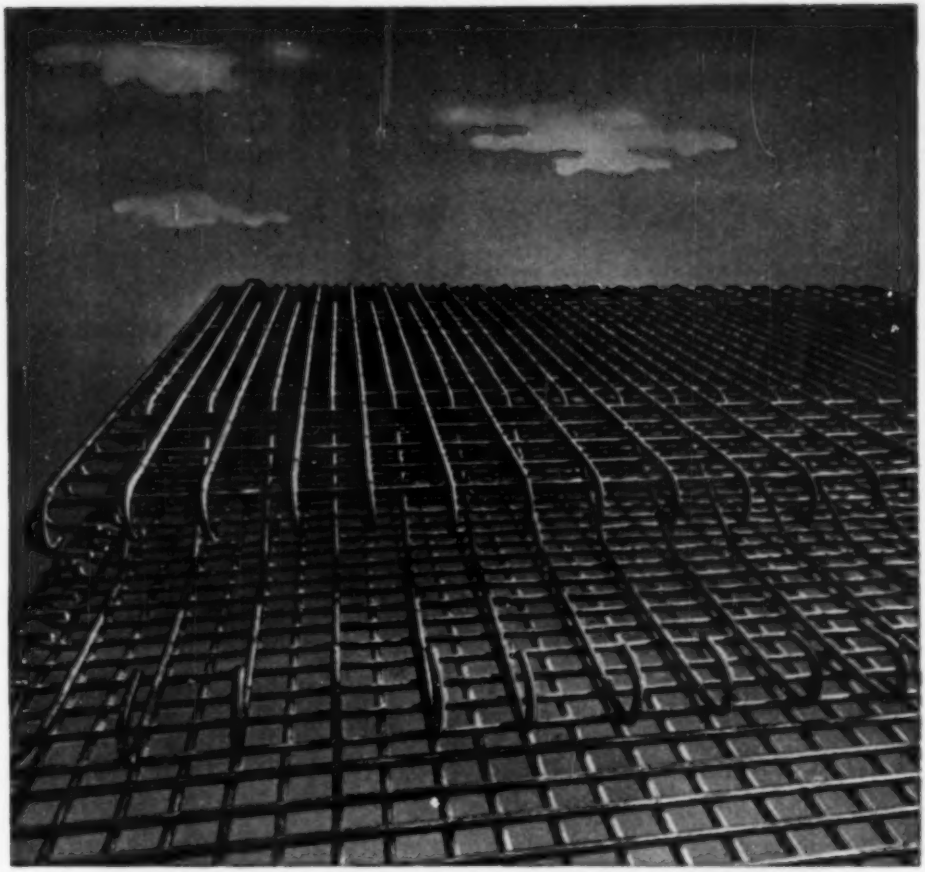

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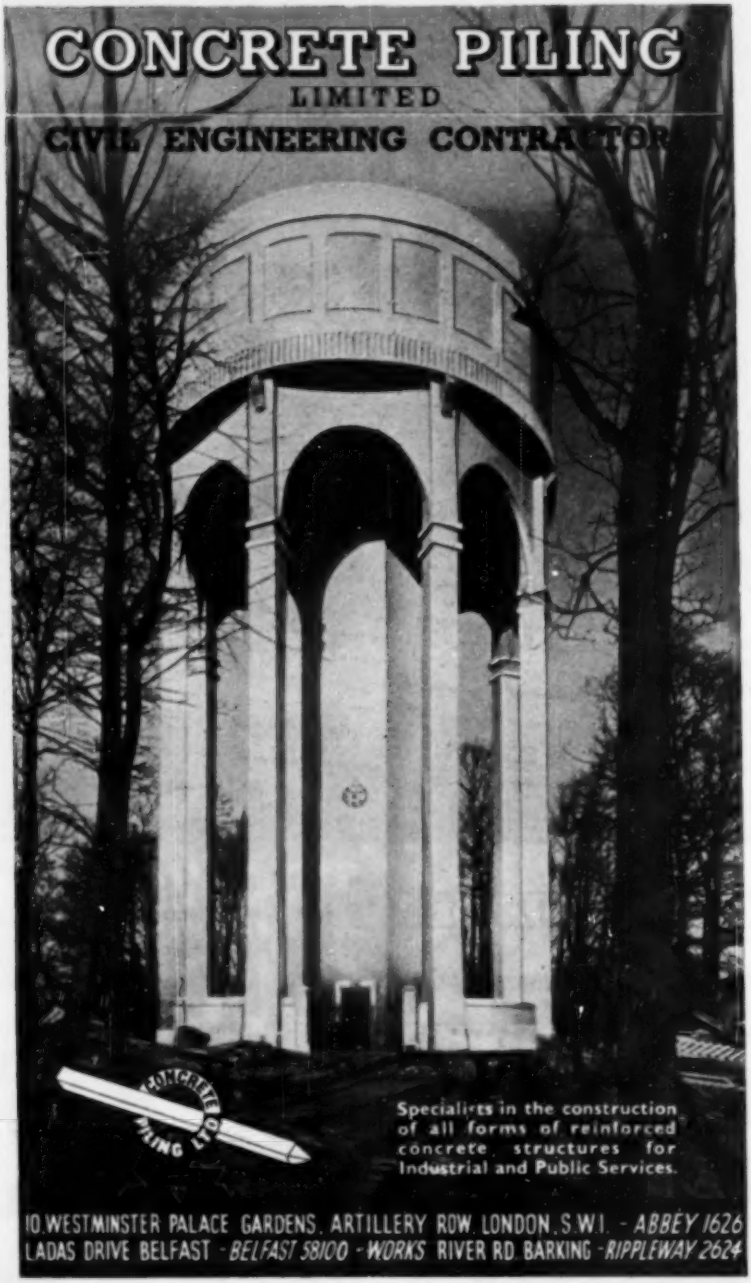
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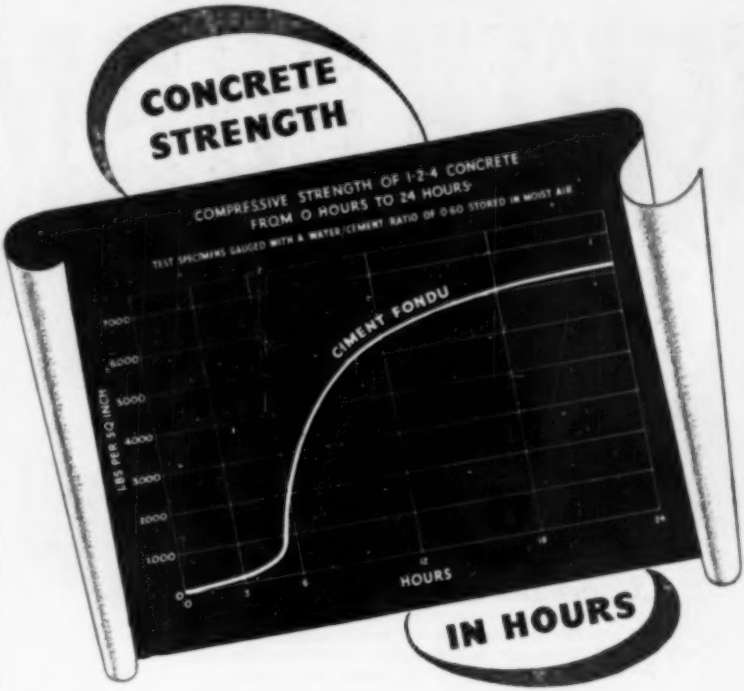

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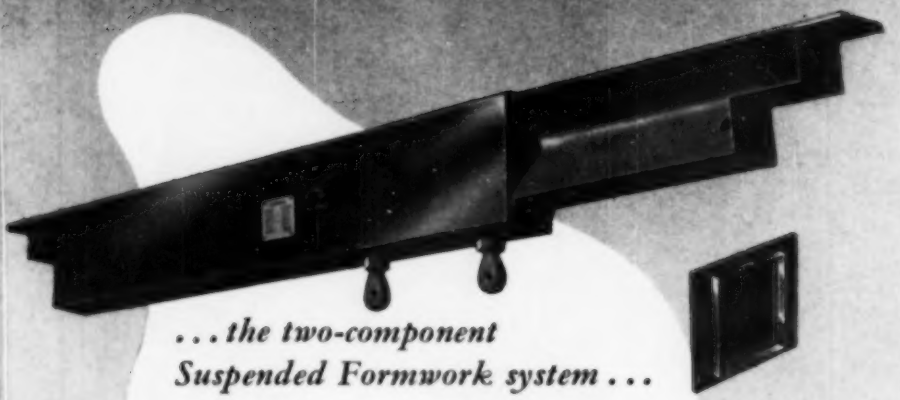

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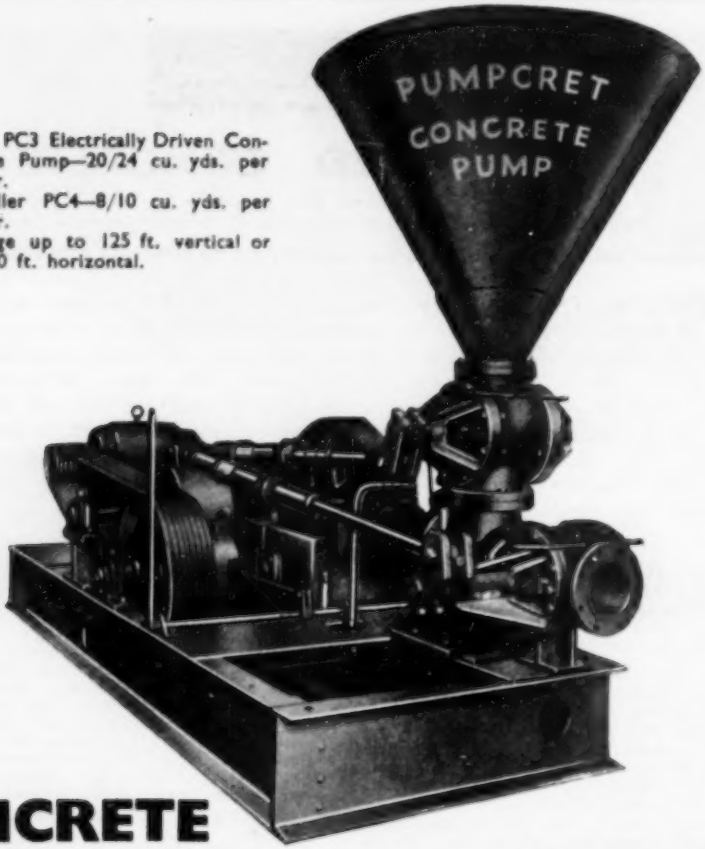

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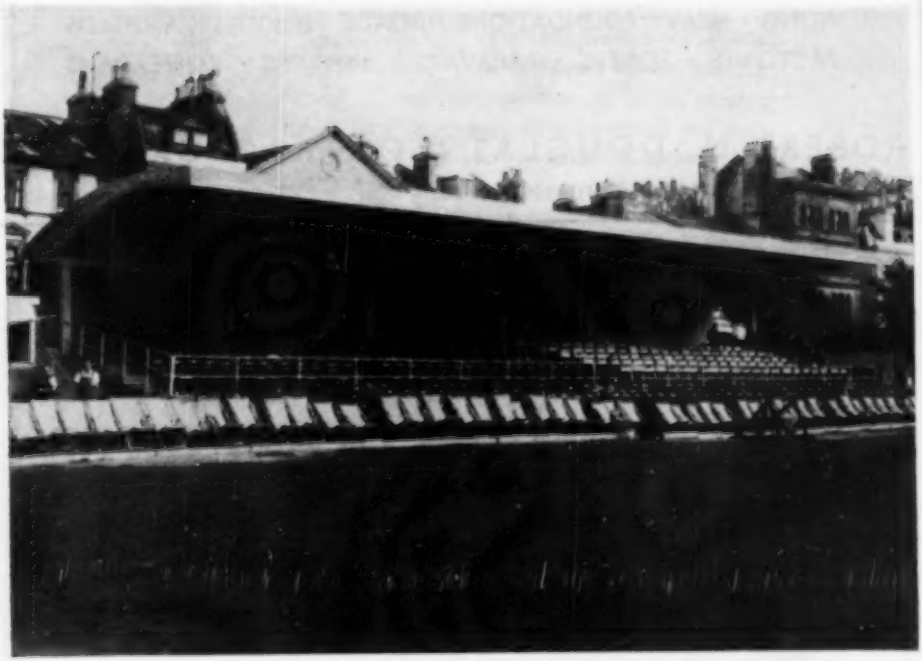

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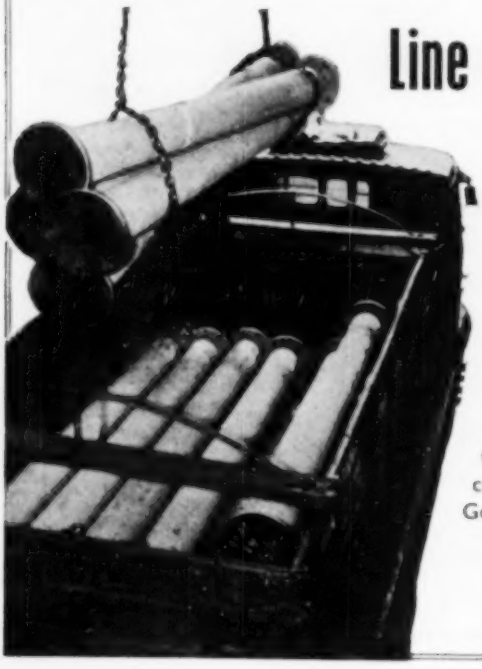

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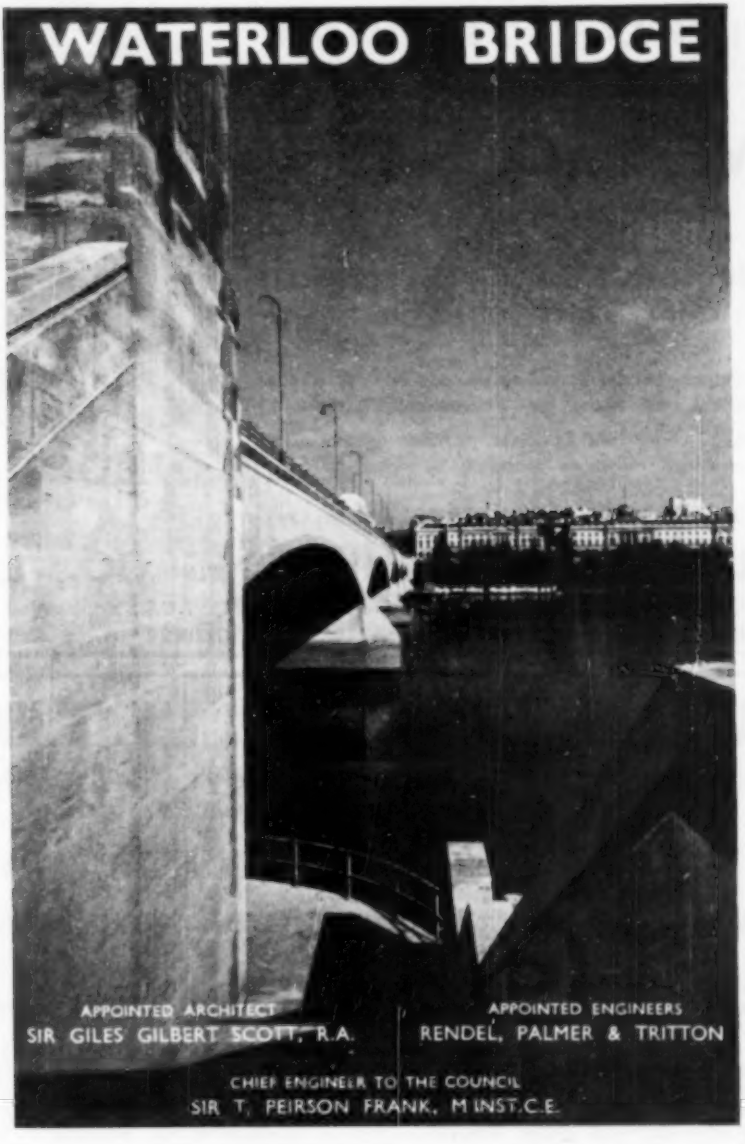

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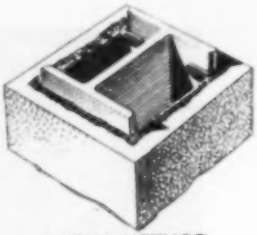

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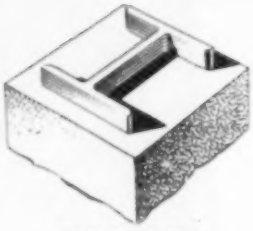

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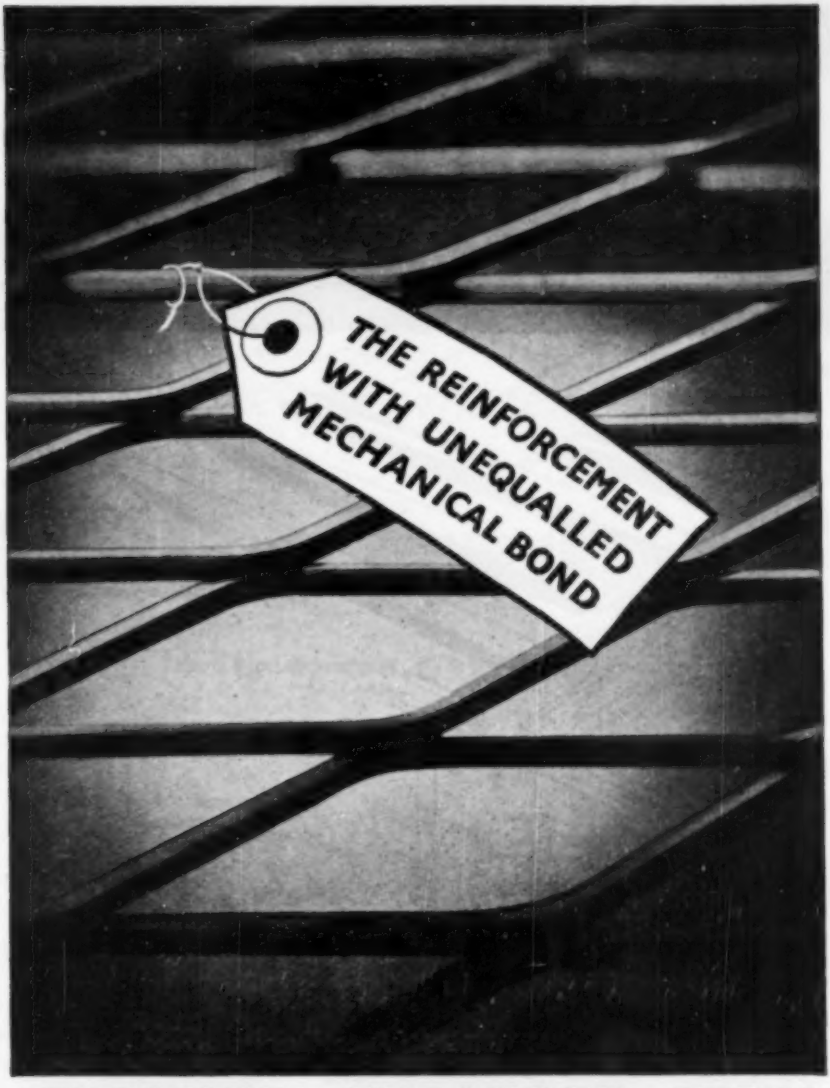

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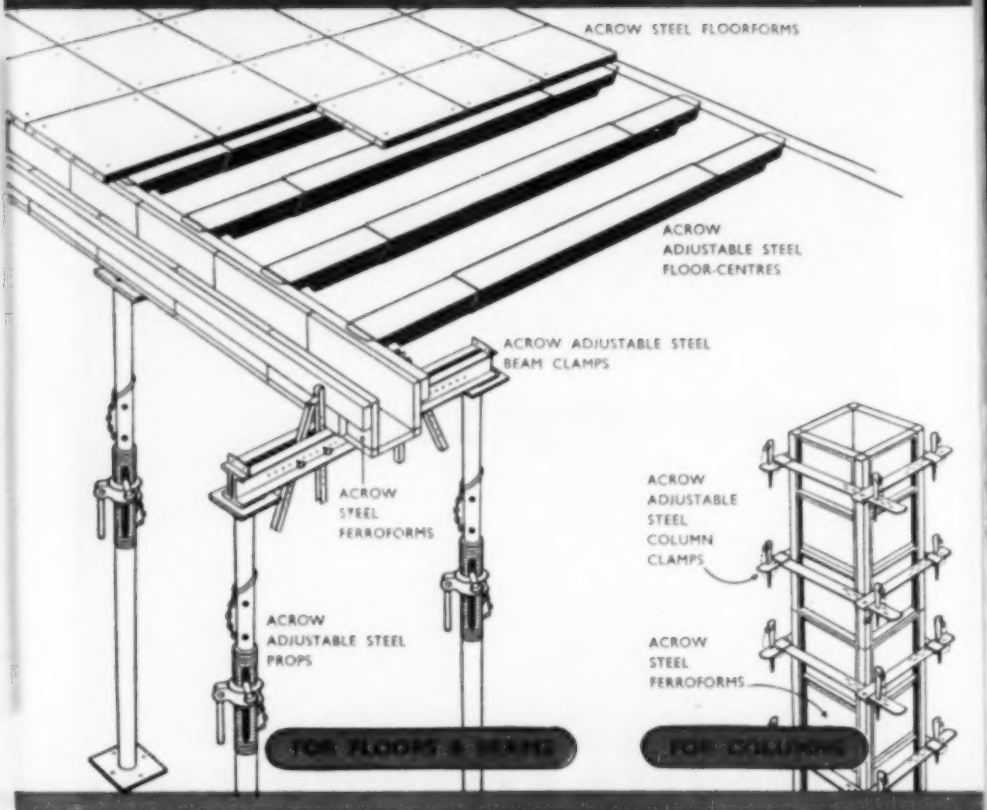

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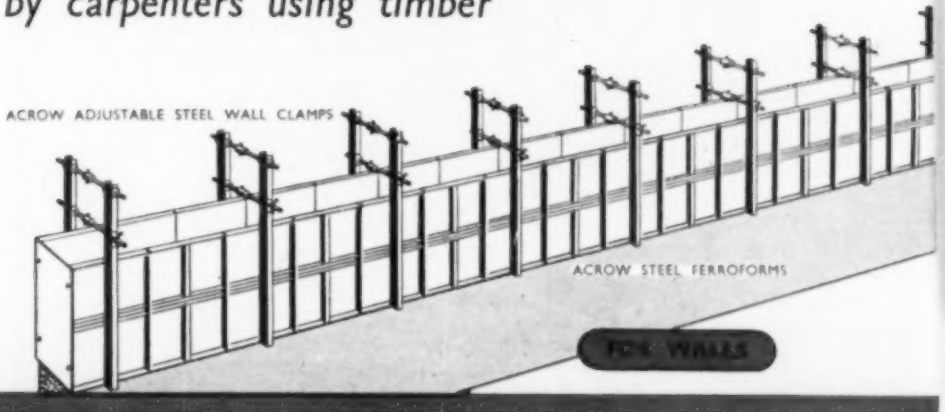

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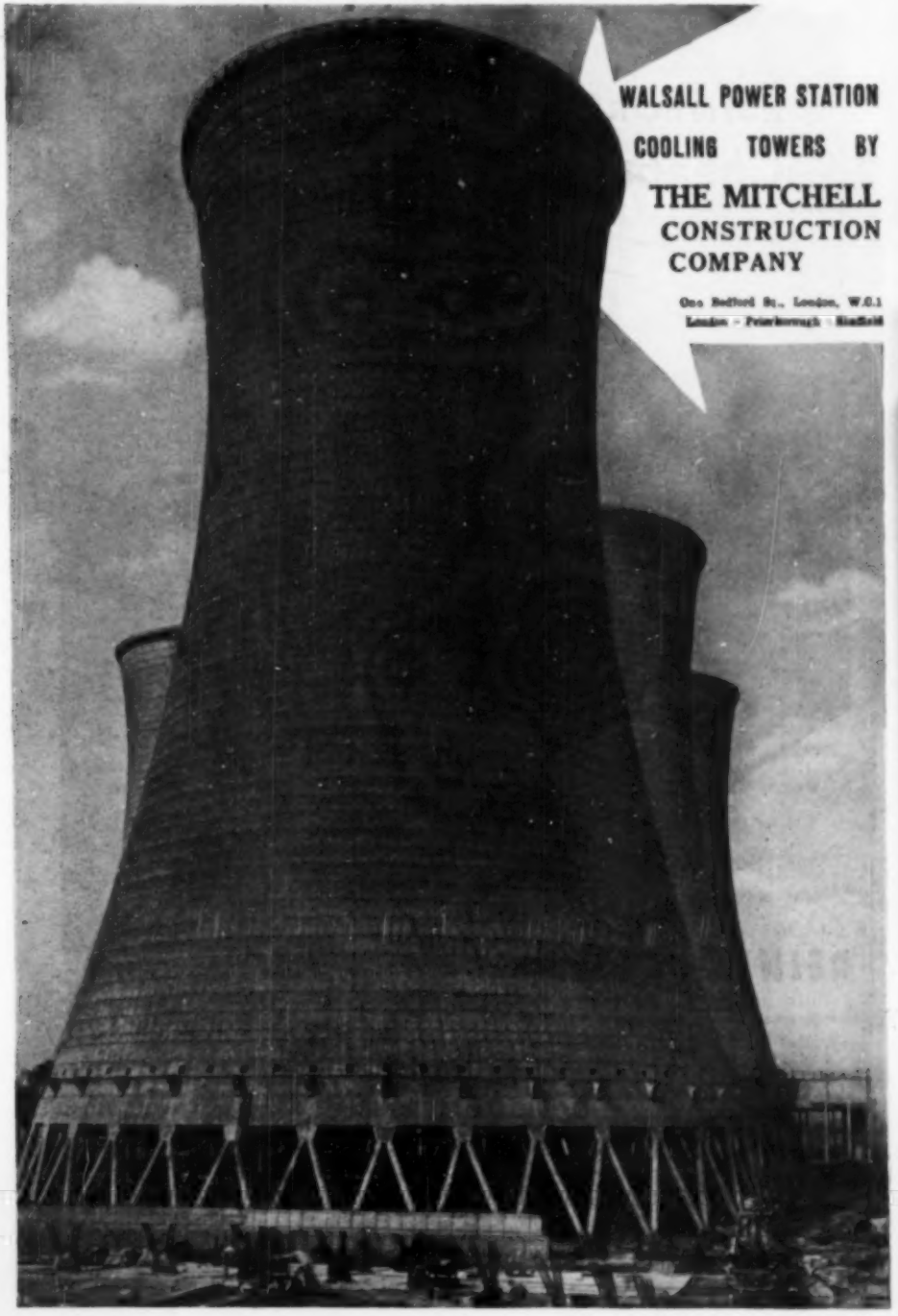
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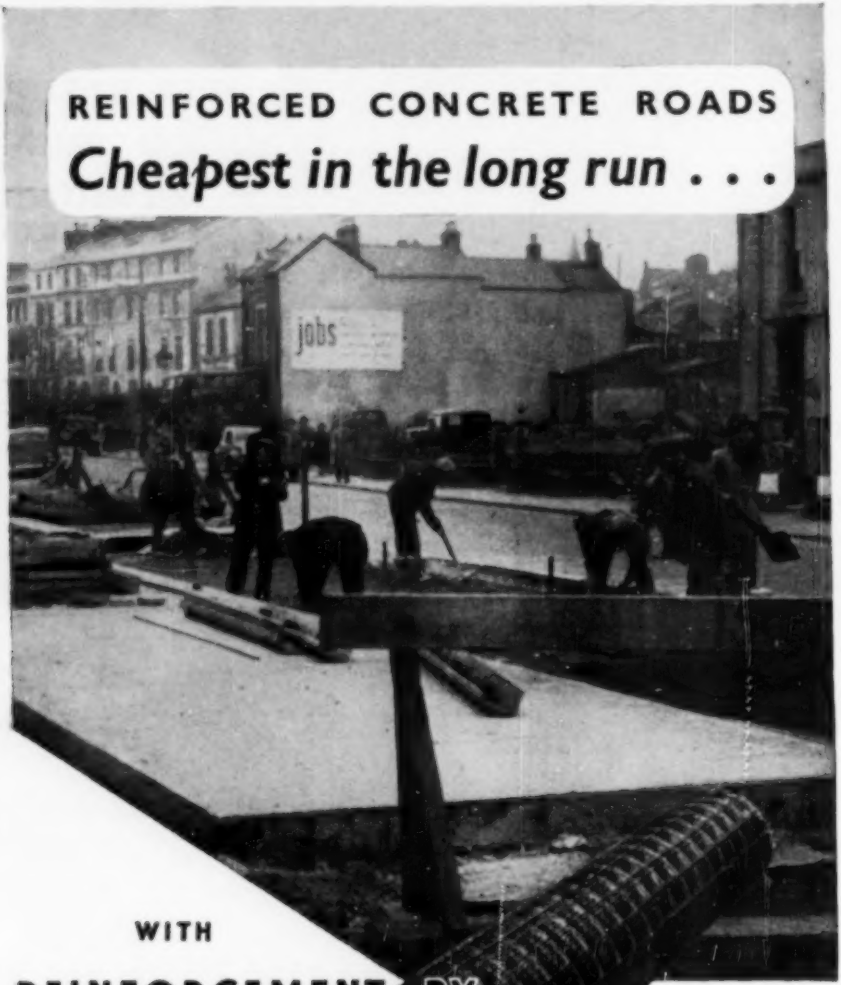

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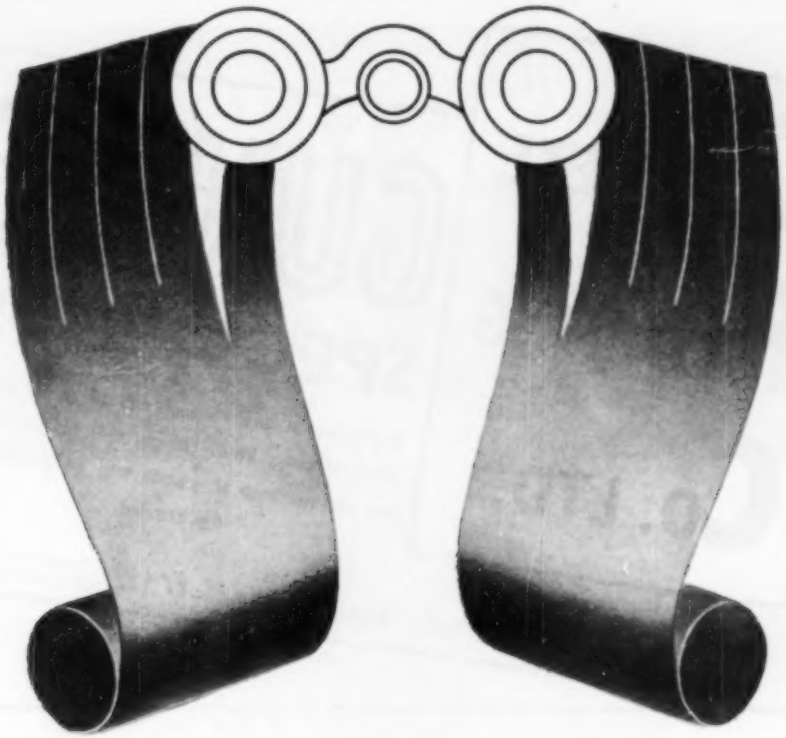

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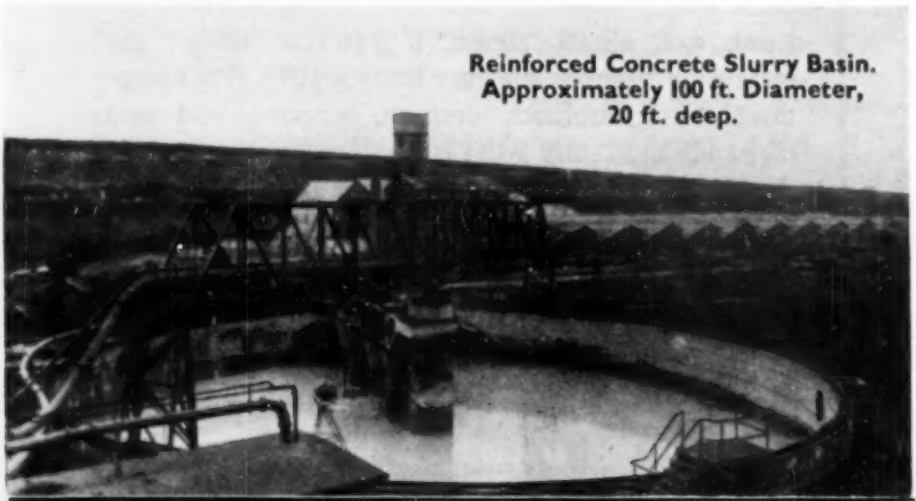

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Volume XLV. No. 11. LONDON, NOVEMBER, 1950

EDITORIAL NOTES

Protection against Atomic Explosion.

THE pressures, and consequent effects, due to explosions are among the less calculable forces to which a structure can be subjected. Experience has indicated the forces due to ordinary high explosives, but the effects of atomic explosions are naturally less well known. Some notes on this subject are given in a booklet entitled " Damage from Atomic Explosion and Design of Protective Structures " recently compiled in the United States of America for the National Security Resources Board by the Department of Defense and the United States Atomic Energy Commission (United States Government Printing Office, Washington 25, D.C. Price 10 cents).

It is stated that the effect on a structure of the initial wind of high velocity emanating from an atomic explosion is like a sudden blow due to the initial rapid rise in pressure, and that this is followed by a steady pressure which lasts until the blast wave has enveloped the structure; the pressure then builds up on the side of the structure farther from the explosion and compresses the structure unless the pressure is relieved by the failure of doors or windows or structural members. This short phase is followed by a wind of less velocity blowing for a much longer time towards the centre of the explosion. The ability of a structure to resist the pressures depends obviously on its strength, to a less degree on its shape, and on the number of openings which may relieve the external pressure.

The damage done by atomic bombs at Hiroshima and Nagasaki was described in this journal for August, 1946. This new U.S. report states that severe damage to reinforced concrete earthquake-resistant buildings does not extend beyond 2000 ft. from an atomic explosion, but within this range all other types of structures are virtually destroyed. At this distance the estimated peak pressure, which endures for about half a second, is 24 lb. per square inch, and is accompanied by a wind of over 500 miles per hour. At 1000 ft. from the explosion the corresponding values are three-eighths of a second, 36 lb. per square inch, and 800 miles per hour, while at one mile they are about one second, 7½ lb. per square inch, and 200 miles per hour. The size of the bomb, weather conditions, and nearness to the explosion obviously affect the degree of damage and the kind of damage. The nearer the explosion the greater is the vertical pressure, causing damaged roofs, and as the distance increases so the horizontal pressure exceeds the vertical pressure. Much secondary structural damage is due to fire.

In considering protective construction, the report makes such obvious suggestions as the dispersal of attractive targets, the prevention of exposure of inflammable materials to radiant heat, and the placing underground of as many services as possible. It is said that concrete walls cast monolithic with the frame of a building should provide a high degree of protection for persons inside the building. Concrete walls 12 in. thick are recommended for the protection of important equipment, and 21 in. thick to protect persons from the effects of radiation from a source half-a-mile distant. It is recommended that protective roofs be designed for a static load of 500 lb. per square foot, which is not so great as to be impracticable when it is considered that warehouse floors are often designed for 336 lb. or more per square foot.

The report also recommends that, to reduce the probability of structural collapse at a distance of half-a-mile from an atomic explosion, a reinforced concrete frame building should be designed to resist a wind imposing a horizontal pressure of 90 lb. per square foot and a vertical pressure of 70 lb. per square foot (which are considerably less than the probable highest pressures), and in general should be designed as an earthquake-resistant structure. The additional rigidity required can be obtained by providing transverse walls and deep lateral beams, and by designing floors and roofs to transmit forces to the transverse walls. Columns of the size ordinarily provided in framed buildings are insufficient to resist the large horizontal forces due to an atomic explosion. Another suggestion is that the building should include redundant structural members, so that if some fail others may not and the building may not collapse. The more ductile a structural material is the more energy it can generally absorb, and for this reason mild steel reinforcement may be better than other types of steel.

The numerical recommendations of the report relate to explosions from atomic bombs that release energy equivalent to that of 20,000 tons of T.N.T. and explode at a height of 2000 ft. Larger bombs will obviously impose greater forces and it is estimated that a bomb of twice this energy would increase the radius of the area in which the damage occurs by 25 per cent. It is, of course, quite unknown what may be the power and proximity of an explosion to which a particular building may be subjected, but, as in designing for any other new conditions, a start must be made somewhere. The data given in this report will therefore probably be used as a first step in the design of resistant structures in spite of the possibility that the atomic and hydrogen bombs now being made, or that will be made in the future, will almost certainly be very different from the bombs exploded over two cities five years ago. It may be remembered that in 1939 data were prepared and used relating to high-explosive bombs weighing from 5 cwt. to 1 ton and for small incendiary bombs of the types then known, neither of which had much relation to the heavy bombs, the " land-mines ", and the phosphorus bombs that were used within a year or two of the outbreak of war, or to the means of providing protection against them. New weapons of war always come as a surprise, and measures of defence lag behind until experience is gained of their effect. But it was found that protection that was thought to be fairly complete against the bombs known in 1939 gave at any rate some protection against the bombs used later, and that it could generally be strengthened and adapted to give a fair degree of safety against more powerful bombs.

Equivalent Elastic Systems in the Analysis of Continuous Structures.—I.

By L. A. BEAUFOY, Ph.D., M.Sc., A.M.Inst.C.E., M.I.Mech.E., M.I.Struct.E., and A. F. S. DIWAN, Ph.D., B.Sc.

In the "Quarterly Journal of Mechanics and Applied Mathematics" for September, 1949, the authors describe how the principle of experimental stress analysis using unloaded models might be applied to obtain an exact analytical solution of hyperstatic two-dimensional continuous structures. The basis of the analysis, which is called the structural analysis, is the evaluation of the elastic properties of the entire structure combined with a study of the way in which it deforms when a deformation is caused at any section, and the results correspond to the immediate results of the experimental procedure. The stress components due to any load on the structure, or alternatively the influence line for any section (see this journal for May, 1950), can be derived from the structural analysis by a subsequent operation called the stress analysis. An alternative method of

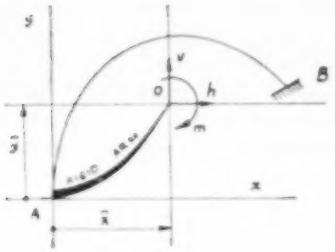

Fig. 1.

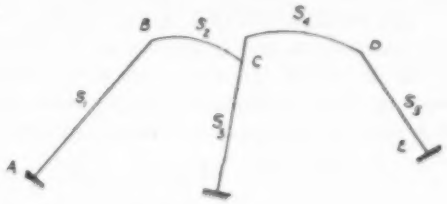

Fig. 2.

evaluating the elastic properties by reducing the whole or a part of the structure to a single fixed-ended system which is elastically equivalent is described in this article, in Part II of which the method will be applied to the solution of a multiple-bay structure, twelve times statically-indeterminate, having flexible columns. For such a structure the usual exact methods of analysis are very laborious. Simultaneous equations and successive approximations are not required, and all calculations can be made with sufficient accuracy on a 10-in. slide-rule.

Moments and rotational displacements are assumed to be positive when clockwise. Horizontal and vertical forces, displacements, and co-ordinate distances are assumed to be positive when measured to the right or upwards.

Relationship between Elastic Constants and Stiffness Factors.

The physical properties of each member of a frame may be expressed in terms of six elastic constants, namely the elastic area $\bar{S}$, the co-ordinates $\bar{x}$ and $\bar{y}$ of the elastic centre, the centroidal moments of inertia I_x and I_y of the member about the x and y axes respectively, and the centroidal product of inertia I_{xy} about the axes x,y. It is shown in the articles mentioned previously that the

following meanings may be attached to the foregoing in relation to a member AB fixed at end B and free at end A (*Fig.* 1):

$\bar{S}$ is the rotation of the rigid arm OA (connecting A to the elastic centre O) due to a unit moment at O (or A).

I_x is the horizontal translation of O due to unit horizontal force at O.

I_y is the vertical translation of O due to unit vertical force at O.

$-I_{xy}$ is the horizontal or vertical translation of O due to unit vertical or horizontal force respectively at O.

It will be shown that the elastic constants for any continuous system of members regarded as a group can be derived from those for each member of the group.

The stiffness factors for one end of a member or for a joint in a structural framework are the horizontal and vertical forces and the moment which are required at that end or joint to produce unit displacement there when the framework carries no load. In the case of the end of a member, the forces and moment are imagined to be applied to a cut at that end. According to the displacement imposed the factors are called horizontal displacement, vertical displacement, or rotational stiffness factors. The elastic constants and the stiffness factors are mutually convertible and this facilitates the combination of a group of structural members and replacement by a simpler system which is elastically equivalent.

Let member AB (*Fig.* 1), the flexural rigidity of which is EI, be acted upon by a moment m and horizontal and vertical forces h and v respectively applied to the end O of the rigid arm OA. Then, as shown in the previous articles, the rotational, horizontal, and vertical displacements ϕ, Δ, and λ respectively at A due to the applied moment and forces at O are

$$\phi = m\bar{S}\,;\quad \Delta = hI_x - vI_{xy} - \phi\bar{y}\,;\quad \lambda = vI_y - hI_{xy} + \phi\bar{x}\,. \qquad (1)$$

Equations (1) express the elastic constants in terms of the displacement coefficients. Consider unit moment applied at A, that is, $m = 1$, $h = 0$, and $v = 0$. The resulting displacements at A are the displacement coefficients for moment, namely,

$$\phi_m = \bar{S}\,;\quad \Delta_m = -\phi_m\bar{y}\,;\quad \lambda_m = \phi_m\bar{x}\,. \qquad (2)$$

The displacement coefficients for horizontal force are obtained by substituting in equations (1) the values $m = -\bar{y}$, $v = 0$, and $h = 1$. Therefore

$$\phi_h = -\bar{y}\bar{S}\,;\quad \Delta_h = I_x - \phi_h\bar{y}\,;\quad \lambda_h = \phi_h\bar{x} - I_{xy}. \qquad (3)$$

Similarly, the displacement coefficients for vertical force are derived by substituting in equations (1) the values $m = \bar{x}$, $v = 1$, and $h = 0$.

$$\phi_v = \bar{x}\bar{S}\,;\quad \Delta_v = -I_{xy} - \phi_v\bar{y}\,;\quad \lambda_v = I_y + \phi_v\bar{x}\,. \qquad (4)$$

Re-arrangement of the foregoing equations yields expressions for the elastic constants in terms of the displacement coefficients. From equations (2),

$$\bar{S} = \phi_m\,;\quad \bar{y} = \frac{-\Delta_m}{\phi_m}\,;\quad \bar{x} = \frac{\lambda_m}{\phi_m}\,. \qquad (5)$$

From equations (3) and (5), and (4) and (5),

$$I_x = \Delta_h - (\bar{y})^2\bar{S}\,;\quad I_{xy} = -\lambda_h - \bar{x}\bar{y}\bar{S}\,;\quad I_y = \lambda_v - (\bar{x})^2\bar{S}\,. \qquad (5a)$$

TABLE I.—ELASTIC PROPERTIES IN TERMS OF STIFFNESS FACTORS.

ELASTIC PROPERTIES (1)	NATURE OF JOINT DISPLACEMENT: ROTATION AND TRANSLATION IN ANY DIRECTION (2)	ROTATION AND VERTICAL TRANSLATION ONLY (3)	ROTATION AND HORIZONTAL TRANSLATION ONLY (4)	ROTATION ONLY (5)
$\bar{S}$	$\frac{A}{AM\phi - BH\phi + CV\phi}$	$\frac{V_\Lambda}{D}$	$\frac{H_\Delta}{E}$	$\frac{1}{M\phi}$
$\bar{x}$	$\frac{C}{A}$	$\frac{V_\phi}{V_\Lambda}$	0	0
$\bar{y}$	$\frac{B}{A}$	0	$\frac{H_\phi}{H_\Delta}$	0
I_x	$\frac{V_\Lambda}{A}$	0	$\frac{1}{H_\Delta}$	0
I_y	$\frac{H_\Delta}{A}$	$\frac{1}{V_\Lambda}$	0	0
I_{xy}	$\frac{H_\Lambda}{A}$	0	0	0

$A = H_\Delta V_\lambda - H_\lambda^2$; $B = H_\phi V_\lambda - H_\lambda V_\phi$; $C = H_\phi H_\lambda - H_\Delta V_\phi$; $D = M_\phi V_\lambda - V_\phi^2$; $E = M_\phi H_\Delta - H_\phi^2$

As shown in the earlier articles, the displacement coefficients are related to the joint stiffness factors by the following:

$$\phi_m = (H_\Delta V_\lambda - H_\lambda^2) \div k \quad . \quad . \quad . \quad . \quad . \quad (6a)$$
$$\Delta_m = -(H_\phi V_\lambda - H_\lambda V_\phi) \div k \quad . \quad . \quad . \quad . \quad (6b)$$
$$\lambda_m = (H_\phi H_\lambda - H_\Delta V_\phi) \div k \quad . \quad . \quad . \quad . \quad (6c)$$
$$\Delta_h = (M_\phi V_\lambda - V_\phi^2) \div k \quad . \quad . \quad . \quad . \quad . \quad (6d)$$
$$\lambda_h = -(M_\phi H_\lambda - H_\phi V_\phi) \div k \quad . \quad . \quad . \quad . \quad (6e)$$
$$\lambda_v = (M_\phi H_\Delta - H_\phi^2) \div k \quad . \quad . \quad . \quad . \quad (6f)$$

in which

$$k = M_\phi(H_\Delta V_\lambda - H_\lambda^2) - H_\phi(H_\phi V_\lambda - H_\lambda V_\phi) + V_\phi(H_\phi H_\lambda - H_\Delta V_\phi),$$

and M, H, and V are the moment, horizontal force, and vertical force respectively acting simultaneously at the joint to produce unit displacement at the joint when all other joints are restrained in the same manner as in the structure, that is, they are the actual stiffness factors for the joints; the subscripts ϕ, Δ, and λ denote the nature of the displacements, that is rotational, horizontal, and vertical respectively.

Combining equations (5), (5*a*), and (6) enables the elastic constants to be xpressed in terms of the stiffness factors, and corresponding expressions are given in column (2) of *Table* No. 1. These expressions apply to the general case in which end A is free to move horizontally and vertically and to rotate. In a continuous frame it is not generally necessary to provide for so many degrees of freedom at the joints, and in columns (3), (4), and (5) of *Table* No. 1 the modified expressions for the respective cases when horizontal movement is prevented, when vertical movement is prevented, and when rotation only is permitted are given.

As already stated, the forces and moment required at end A of a member AB fixed at B (*Fig.* 1) to produce unit displacement at A in a given direction are

equal to the stiffness-factors which, it can be shown, can be expressed in terms of the elastic constants for AB as in equations (7) in which

$$I'_x = I_x - \frac{I^2_{xy}}{I_y}, \text{ and } I'_y = I_y - \frac{I^2_{xy}}{I_x}.$$

$$\left.\begin{aligned} m_\phi &= \frac{1}{S} + \frac{(\bar{y})^2}{I'_x} + \frac{(\bar{x})^2}{I'_y} - \frac{2_{xy}I_{xy}}{I'_xI_y} \\ h_\phi = m_\Delta &= \frac{\bar{y}}{I'_x} - \frac{\bar{x}I_{xy}}{I'_xI_y} \\ v_\phi = m_\lambda &= \frac{\bar{y}I_{xy}}{I'_yI_x} - \frac{\bar{x}}{I'_y} \\ h_\Delta &= \frac{1}{I'_x}; \quad v_\Delta = h_\lambda = \frac{I_{xy}}{I'_xI_y}; \quad -v_\lambda = \frac{1}{I'_y}. \end{aligned}\right\} \quad . \quad . \quad (7)$$

Combination of Elastic Systems.

In the general case of a continuous frame, adjoining members can be combined in series or in parallel. In *Fig.* 2 members S_1 and S_2 may be combined in series to form one elastic system between A and C, while S_1 and S_2 together may be combined in parallel with S_3 at C to form one elastic system which is then in series with S_4 and S_5. The determination of the elastic constants for such combinations is considered in the following.

Two or more elastic members are in series with reference to the outer terminals when they are successively connected to one another so as to form one elastic system. Consider two members S_1 and S_2 (*Fig.* 3*a* and *b*). Let the elastic constants for S_1 be $\bar{S}_1$, I_{x1}, I_{y1}, and I_{xy1}, and the elastic centre O_1, and for S_2, $\bar{S}_2$, I_{x2}, I_{y2}, I_{xy2}, and O_2. Now suppose that S_1 and S_2 are connected in series so that B_1 and A_2 coincide and make the continuous system $A_1B_1B_2$ (*Fig.* 3*c*). Now, a load which is external to the system can act on it only through the terminals A_1 and B_2, and is transmitted wholly from S_1 to S_2, so that the internal forces in both members are those due to the whole external load. This is also true of forces transmitted to the system from a member, connected to either

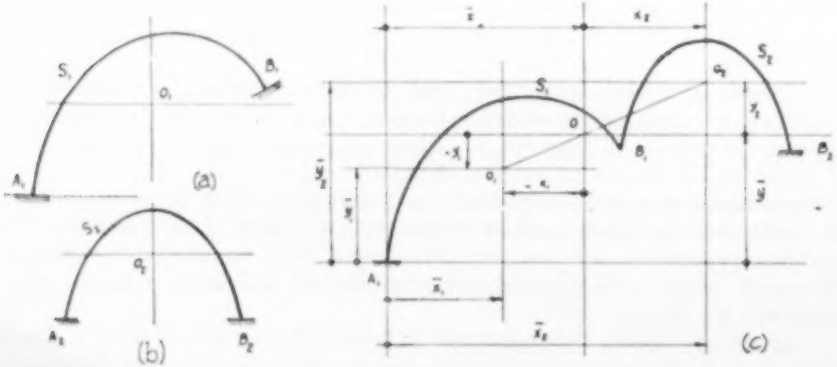

Fig. 3.

A_1 or B_2, in which a relative deformation has been imposed at any cut section. The elastic area $\bar{S}$ for the system is obtained as the sum of the elastic areas of each member, while the position of the centroidal axes of the system with reference to one end, say A_1, is found by combining the elastic areas of the two members by the ordinary processes of mechanics. Centroidal values of I_x, I_y, and I_{xy} for the combination follow. Therefore

$$\left.\begin{aligned} \bar{S} &= \bar{S}_1 + \bar{S}_2 \\ \bar{x} &= \frac{\bar{S}_1\bar{x}_1 + \bar{S}_2\bar{x}_2}{\bar{S}}; \quad \bar{y} = \frac{\bar{S}_1\bar{y}_1 + \bar{S}_2\bar{y}_2}{\bar{S}} \\ I_x &= I_{x1} + I_{x2} + \bar{S}_1 y_1^2 + \bar{S}_2 y_2^2 \\ I_y &= I_{y1} + I_{y2} + \bar{S}_1 x_1^2 + \bar{S}_2 x_2^2 \\ I_{xy} &= I_{xy1} + I_{xy2} + \bar{S}_1 x_1 y_1 + \bar{S}_2 x_2 y_2 \end{aligned}\right\} \quad . \quad . \quad . \quad (8)$$

Equations (8) must be interpreted with regard to sign. In the case considered, x_1 and y_1 are negative.

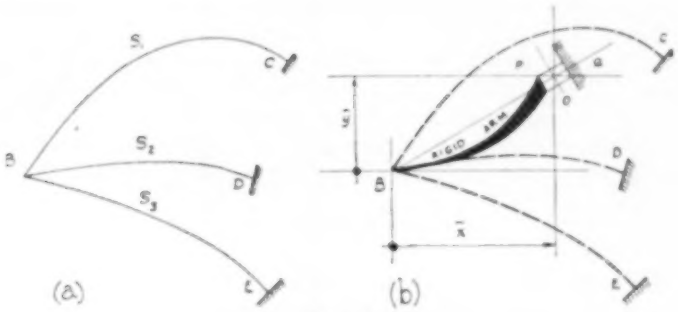

Fig. 4.

For a combination of structural members in parallel the elastic constants of the members cannot be combined directly to give the elastic constants for the combination, and it is convenient in this case to use the stiffness factors. If two or more elastic members meet at a common joint B (*Fig. 4a*) they are in parallel with reference to the joint. An external moment or force acting at the joint is resisted by all the members connected at the joint, and may be transmitted either from an actual load carried by another member (not shown in the diagram) framing into the joint B or from a relative deformation at a cut in such a member. The internal forces in each member are those due to the share taken by each member in the total resistance of the group. Consider the members S_1, S_2, and S_3 connected in parallel at joint B. The ends C, D, and E may be fixed, as shown, or they may be continuous with other members not shown in the diagram. The actual stiffness factors of the joint B are the sums of the relevant actual stiffness factors of the members connected at the joint, and substitution of the values of these factors in the expressions in *Table* No. 1, according to the nature of the joint displacement permitted by the conditions, gives the elastic constants of the single elastic system which behaves in the same manner as the system S_1, S_2, and S_3.

Virtual Equivalent System.

The single elastic system which replaces a combination of members in series or in parallel, or both, as in the foregoing, is not a real system, and is called a virtual equivalent system, which it is convenient to represent diagrammatically as BPQ (*Fig.* 4*b*) and which replaces the combination of members in *Fig.* 4*a*. The elastic area of the virtual equivalent system is represented by PQ, which is

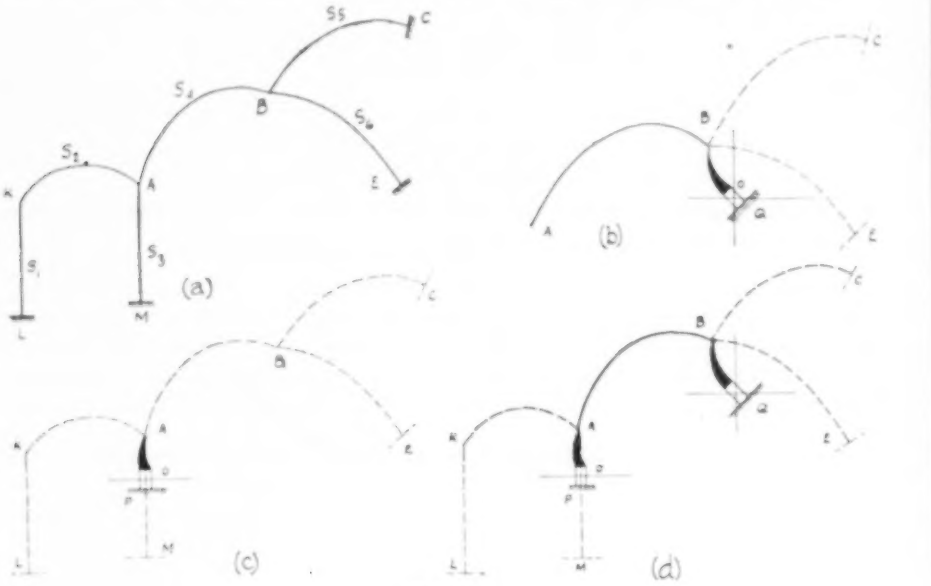

Fig. 5.

considered fixed at Q and connected by a rigid arm PB to joint B; its elastic centre O is at horizontal and vertical distances $\bar{x}$ and $\bar{y}$ respectively from joint B. No moments or forces may be considered to be applied to a virtual equivalent system at a point between its terminals; the only load it may be imagined to carry is that due to external moments or forces transmitted to it through its terminals. A virtual equivalent system may be used to replace a part or the whole of a structure.

Partial Equivalent System.

Any part of a continuous frame may be reduced to a compound system composed of one real loaded member (or member in which there is to be a relative deformation at some section) and a virtual equivalent system, the two being connected in series. Such a compound system is a partial equivalent system. Consider the continuous frame of *Fig.* 5*a*. Imagine the member AB to be completely severed at the joint A (*Fig.* 4*b*). Then the part of the frame to the right of the cut can be reduced to the real member AB linked in series with a virtual equivalent system BQ which replaces members BC and BE. The system ABQ constitutes a partial equivalent system, for which the elastic constants are readily determined. If the values of these constants are substituted in

equations (7), the values of the stiffness factors for the end A of the member AB can be calculated.

If a moment and forces equal respectively to the stiffness factors for the end A are applied at A, the end A by definition undergoes unit displacement in the direction corresponding to the stiffness factors used. If we transfer the forces and the moment to the elastic centre O of the virtual equivalent system, making the necessary correction to the moment, and apply equations (1), the resulting displacements are the movements induced at joint B.

Full Equivalent System.

The whole of a continuous frame may be reduced to a full equivalent system comprising either a single virtual equivalent system associated with a particular joint of the frame, or one real member connected in series with one or more virtual equivalent systems. The structure in *Fig.* 5*a* may be reduced, for example, to a full equivalent system AP (*Fig.* 5*c*) associated with the joint A, in which case the full equivalent system replaces entirely all members of the structure. Alternatively it may be reduced to a full equivalent system composed of a real member AB together with the virtual equivalent systems AP and BQ (*Fig.* 5*d*).

It follows from a previous paragraph that the only forces that may be considered to act on the full equivalent system AP (*Fig.* 5*c*) are the moment and forces at the joint A together with their reactions at the support P. If we transfer these to the elastic centre O, making the necessary correction to the moment, and apply equations (1), the resulting displacements are the total joint displacements at A.

(To be concluded.)

The Liberal Arts and Engineering.

THE increasing importance attached to the more liberal education of students of engineering in the United States of America is indicated by the following abstract from "Engineering News-Record."

The Case Institute of Technology, Cleveland, Ohio, has entered into an arrangement with five other middle-western colleges to inaugurate a new curriculum for combined arts and engineering education. This arrangement, called the "Binary Curriculum," requires three years of study at one of the liberal arts colleges and two years and a summer at the Case Institute of Technology. On graduating at each institution, students will receive bachelor of arts degrees from the arts colleges at which they have completed their first three years, and bachelor of science degrees in engineering from the Case Institute of Technology.

A programme enabling engineering students to obtain a broad general education before concentrating on a special branch of science or technology has been arranged by the Rensselaer Polytechnic Institute and Franklin and Marshall College. Under this plan, students complete three years of the regular liberal arts programme at Franklin and Marshall College, with emphasis on the basic sciences and mathematics, before entering on two years of specialized study at the Rensselaer Polytechnic Institute. After satisfactorily completing these courses, the students will receive an arts degree from Franklin and Marshall College and a bachelor's degree in engineering at Rensselaer Polytechnic Institute.

A similar arrangement has recently been made by the Rensselaer Polytechnic Institute and three other centres of the liberal arts, namely, the Washington and Lee University, Trinity College, and St. Lawrence University.

Book Reviews.

"Explanatory Handbook on the British Standard Code of Practice for Reinforced Concrete." By W. L. Scott, W. H. Glanville, and F. G. Thomas. (London: Concrete Publications, Ltd. 1950. Price 9*s*.)

THIS book is the successor to the explanatory handbook, written by two of the present authors, on the Code of Practice issued by the Department of Scientific and Industrial Research in 1934, and in style it follows the earlier work. Each clause of the British Standard Code (CP. 114—1948), "The Structural Use of Normal Reinforced Concrete in Buildings", is printed in full, and is followed by comments on the reasons for, and the application of the recommendations contained in, the clause. In addition to the tables and illustrations given in the Code, the authors have prepared thirty-seven tables and eighteen diagrams to aid the designer and constructor in the use of the Code. The subject matter is subdivided as in the Code, that is: Scope and definitions; Materials and properties of concrete; Design; Workmanship; Inspection, testing, and maintenance. The appendixes give standard tests for determining the consistency and strength of concrete and the bulking of fine aggregate. The design section occupies about two-thirds of the book and deals with stresses, spacing of bars, stiffness, bond, beams and slabs, columns, flat slabs, walls, and fire resistance. The authors' tables in this section relate to bond lengths and bends of bars, distribution bars in slabs, bending moments on beams continuous over equal spans, resistance-moment factors, slabs spanning in two directions, ties in columns, permissible loads on square and octagonal columns, frame constants and fixed-end bending moments, moments of inertia of tee-beams, and bending moments on and effective depths of flat slabs. The authors' tables in the section on work on the site include the effect of temperature on the strength of concrete, the minimum thickness of concrete to be placed in frosty weather, and times for the removal of shuttering.

"Fortschritte im Stahlbeton." By Rudolf Saliger. (Vienna: Franz Deutike. 1950. 3.20 dollars.)

IN this book of 138 pages progress in the use of steel reinforcement of high strength is surveyed, and the consequent development of plastic theories, a conception which the author claims to have first introduced in 1936, is described. Twenty-five years ago the stresses in mild steel reinforcement seldom exceeded 17,000 lb. per square inch, whereas steel now used may have a yield-point stress of 70,000 lb. per square inch and a working stress exceeding 30,000 lb. per square inch. The permissible stress in reinforcement is not unlimited, however, having regard to the unavoidable cracking of concrete which, if uncontrolled, may result in a permanent weakening of the structure because moisture may reach and corrode the reinforcement. The width of crack which will not adversely affect a structure has not yet been determined with certainty, but the author suggests that cracks not exceeding 0·008 in. wide for protected buildings and 0·004 in. wide for exposed structures appear to be admissible.

"Plant for Prestressing Concrete." (London: H.M. Stationery Office. 1950. Price 1*s*. 3*d*.)

IN this brochure of 16 pages are illustrated and described the methods of tensioning the wires in prestressed concrete used at the experimental station of the Ministry of Works. Various styles of wedges and chucks for retaining the ends of the wires, anchor plates, jacks, quick-release grips, and moulds are clearly illustrated. The brochure will be of interest and use to those concerned with prestressed concrete.

"Concrete Practice in Building Construction." (London: Cement and Concrete Association, 52 Grosvenor Gardens, London, S.W.1.)

THIS booklet has been prepared for the use of clerks of works, foremen, and others engaged on the supervision of concrete work. It deals with materials and constructional methods, and gives some memoranda. Copies will be sent free on request.

"Laxton's Builders' Price Book." (London: Kelly's Directories, Ltd. 1950. Price 30*s*.)

THIS well-known volume is again available after a lapse of ten years. It has been brought up to date and revised by Mr. P. J. Walters, F.R.I.C.S., and is as valuable as its predecessors to those concerned with preparing specifications and estimating for building work.

Book Received.

"Report on the Structural Use of Aluminium Alloys in Buildings." (London: Institution of Structural Engineers. 1950. Price 3*s*. 6*d*.)

Testing the Quality of Concrete in a Thick Wall.*

By R. JONES, Ph.D., B.Sc.
(ROAD RESEARCH LABORATORY, DEPARTMENT OF SCIENTIFIC AND INDUSTRIAL RESEARCH.)

NON-DESTRUCTIVE tests have been made on the concrete walls, 3 ft. 6 in. and 4 ft. 3 in. thick, of a reservoir. An ultrasonic pulse was transmitted through the wall and its velocity of propagation used to ascertain the variations in the quality of the concrete placed at different heights in various parts of the walls. The comparative velocities are converted into equivalent strengths by taking similar measurements on cubes of concrete of the same mixtures. The results show that the concrete placed in the walls is inferior in quality to that in the test cubes at the same age. There were often large variations in the quality of a concrete in one lift, particularly if the height of the lift was about 2 ft., while in lifts of only 9 in. the quality was higher and more uniform. Improvements in the quality of the concrete mixture produced better concrete in the wall. The tests and the conclusions drawn therefrom are described in the following.

Previous work [1] at the Road Research Laboratory has established that the velocity of propagation of an ultrasonic pulse through a specimen of concrete is directly related to the compressive strength of the concrete, and that such measurements may be used to determine the strength of concrete in structural members.[2]

In the initial stages of construction of the walls of the reservoir it was noticed that some of the concrete cubes had abnormally low compressive strengths. The Road Research Laboratory was asked to use its method of non-destructive testing to determine the equivalent strength of the concrete at selected positions in the walls, and to discover to what extent the strength of the earlier cubes was representative of the strength of the concrete in the walls. The walls of the reservoir were constructed in three lifts. Measurements of the bottom and second lifts, where the wall is 4 ft. 3 in. and 3 ft. 6 in. thick respectively, were made. The concrete comprised ordinary Portland cement, sand complying with B.S. No. 882, and graded rounded gravel aggregate (also complying with B.S. No. 882) in the proportions of 1 : 2½ : 5½ by volume. The cement was fed from a hopper into a 1-ft. cube measuring box for the first few batches but the packing was loose; subsequently the volume of cement was increased and steps were taken also to make more constant the weight of the cement. The water-cement ratio was at first 0·66 but was later reduced to 0·63. The walls were thus made of concrete which varied in quality. The wet concrete was placed by crane and bucket and spread by hand between steel shuttering. The concrete was not vibrated, and the only tamping it received was treading by the workmen engaged in spreading it.

The main details of the method of testing are as follows. Two piezo-electric crystal transducers were held in contact with the concrete on opposite sides of the wall. Adequate contact was achieved by applying a film of oil to the surface of the wall after it had been gently rubbed with a wire brush to remove surface deposits of cement and mortar. A short train of mechanical vibrations was

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produced by electrically shock-exciting one of the transducers. The vibrations passed through the concrete and were picked up by the second transducer. The electrical signal from this transducer was amplified and applied to a pair of the plates of a double-beam cathode-ray tube to produce a visual signal on one of the traces. Accurately-known timing marks were produced on the second trace at intervals of 1·017, 10·17, and 101·7 micro-seconds by means of a selector switch. The time taken for the leading edge of the pulse to travel through the concrete was obtained from observation of the movement of the visual signal relative to the timing marks measured from a zero position when the faces of the transducers

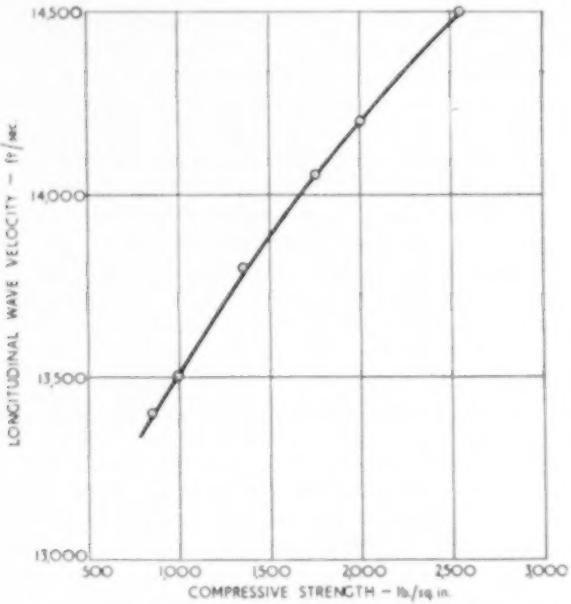

Fig. 1.—Relation between Longitudinal-Wave Velocity and Compressive Strength.

were in contact. The longitudinal-wave velocity, that is the velocity of propagation of the leading edge of the pulse, was calculated from the observed time of transmission and the measured thickness of the wall. This method is essentially that used in the previous work on laboratory specimens [1] and the only modifications to the apparatus used on the reservoir walls were the provision of increased sensitivity and the facility to measure longer transmission times.

The measurements of the longitudinal-wave velocity, while providing information of the variations in the quality of the concrete, require to be translated into equivalent cube strengths to provide a criterion of quality. To do this a relation was obtained between the longitudinal-wave velocity and the compressive strength at different ages of a number of concrete cubes of the concrete having a water-cement ratio of 0·63 used in the construction of the walls. The relation was slightly modified, as a result of further laboratory tests, to allow for the drying which occurred in the walls. The curve used to convert the observed

longitudinal-wave velocity in the walls to equivalent cube strength is given in *Fig.* 1.

The results of the tests are given in *Table* I, and comprise: (*a*), The mean longitudinal-wave velocity through the concrete in each lift (V); (*b*), The estimated cube strength corresponding to the mean longitudinal-wave velocity (W); (*c*), The estimated mean strength of the concrete in each lift at 28 days (S_w); and (*d*), The actual or estimated strength of test cubes at 28 days (S_c). The estimated values in (*c*) and (*d*) were obtained from strength-age curves for various concretes made at the Road Research Laboratory in the course of other investigations.[3]

Table I shows that the concrete in the walls is inferior to that in cubes of the same age. In most cases the concrete in the walls took 28 days or more to reach a strength equal to that of the cubes at seven days, which suggests

TABLE I.—RESULTS OF TESTS ON RESERVOIR WALLS—COMPARISON OF MEAN STRENGTH OF WALL AND STRENGTH OF CUBES AT 28 DAYS.

Wall	Thickness of wall	Lift of concrete	Depth of lift (in.)	Tests on wall			S_w (lb. per sq. in.)	S_c (lb. per sq. in.)	$\frac{S_w}{S_c}$
				Age when tested (days)	V (ft. per second)	W (lb. per sq. in.)			
No. 1	3 ft. 6 in.	Top	18	26	14,050	1750	1750	4250	0·42
		Bottom	24	32	13,800	1350	1300	2950	0·44
	4 ft. 3 in.	Top	24	39	13,500	1000	950	1500*	0·61
		5th	9	40	14,050	1750	1700	2500	0·68
		4th	12	61	13,400	850	800	1550*	0·52
		3rd	12	62	13,500	1000	950	2000*	0·47
		2nd	12	62	13,500	1000	950	2000	0·47
		1st	12	67	13,600	1100	1050	1800*	0·57
No. 2	4 ft. 3 in.	Top	16½	33	13,800	1350	1300	1900*	0·68
		3rd	16½	34	13,850	1400	1350	2350	0·57
		2nd	24	60	13,950	1550	1450	3500*	0·42
		1st	24	63	13,400	850	800	2250	0·36
No. 3	4 ft. 3 in.	3rd	9	13	14,500	2500	3100	4550	0·68
		2nd	24	20	13,800	1350	1500	3250	0·46
		1st	24	25	14,700	2000	2050	3850	0·53

* Estimated from strength at 7 days.

that the concrete in the walls has a lower density than that in the cubes, probably due to poor tamping. It is seen that, in general, changes in the quality of the concrete in the walls are similar, if not proportional, to changes in the quality of the cubes, a result that may have been expected, since there was no appreciable variation in the slumps and the workability of the concrete should have remained fairly constant. Under such conditions it is expected that an improvement in the quality of the concrete in the walls would be obtained by using a higher cement content. There was, however, an increase in the slump when the cement content was initially increased, so that the water-cement ratio was reduced from 0·66 to 0·63 to produce a further improvement.

The last column in *Table* I gives the ratio of the strength of the concrete in the walls to that of the corresponding cube. There is a considerable variation

in this ratio, but examination shows that it is related in some degree to the height of the lift, the mean values indicating that better compaction was obtained by placing the concrete in smaller lifts. The reduction in the quality of the deeper lifts was generally due to the variation of quality with height. The bottom of the lift invariably contained concrete in which the longitudinal-wave velocity was much higher than that in the concrete at the top. This phenomenon can be due either to separation of the coarse aggregate or, more likely, to the presence of more water or air voids near the top. In support of the latter suggestion, there was porosity near the top of some of the deeper lifts in contrast to the smooth surface generally obtained.

The main conclusions from the test are as follows: (1) The concrete in the cubes was much stronger than the concrete in the walls at the same age. (2) There was a similar, if not proportionate, variation between the strengths of the concrete in the cubes and the concrete in the walls. Thus when the quality of the mixture was improved by increasing the cement content there was an increase in the strengths of the cubes and the concrete in the walls. (3) Placing the concrete in 9-in. lifts produced better concrete than in 24-in. lifts. The deeper lifts tended to give concrete of poorer quality with some porosity near to the top of the lift.

The work described in this paper was carried out by the Road Research Laboratory of the Department of Scientific and Industrial Research as part of the programme of the Road Research Board. The paper is published by permission of the Director of Road Research.

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(2) Jones, R. A non-destructive method of testing concrete during hardening. *Concr. Constr. Engng.*, 1949, **44** (4), 127–9.
(3) Collins, A. R. Proportioning concrete for strength and workability. *Concr. Constr. Engng.*, 1939, **34** (10), 557–68.

International Association for Bridge and Structural Engineering.

THE fourth congress of the International Association for Bridge and Structural Engineering will be held at Cambridge in August 1952. The subjects to be discussed include (1) The bases of the calculations of the safety of structures; (2) Fundamental principles of the properties of concrete; (3) Current problems of design and construction in concrete, reinforced concrete, and prestressed concrete. Further information may be had from the Secretary of the Association, at the Federal Institute of Technology, Zürich, Switzerland.

Rapid Construction of Silos.

AN average height of 19 ft. 6 in. was concreted daily with continuously-moving forms in the construction of the walls of a structure comprising twenty-four reinforced concrete silos 127 ft. high near Columbus, Ohio. The bins are 20 ft. in diameter, and the walls are generally 7 in. thick, the external walls being 8 in. thick. The concrete (6300 cu. yd.) was deposited in seven days; it was mixed in transit from batching plants five miles and eight miles distant. Six 3-cu. yd. truck-mixers were required when the concrete was brought from the nearer plant, and ten when from the farther plant which was used at night.

The Roof of the Royal Festival Hall, Festival of Britain.

THE design of the superstructure and the construction of the foundations and lower parts of the concert hall being erected for the London County Council on the site of the Festival of Britain are described in this journal for January, 1950. In the following is described the roof of the hall, which comprises two reinforced concrete slabs with a space of about 2 ft. between them and, as shown in *Fig.* 2, a 2-in. glass-wool blanket placed in the cavity to provide acoustical insulation. The top of the upper slab is covered with insulating board and sheet copper. The lower slab is 8 in. thick and is of composite precast and cast-in-situ construction. Precast concrete beam soffits, 5 in. by 2 in., in each of which is embedded a welded steel lattice, are placed at 21 in. centres between the steel trusses which are at 12-ft. 6-in. centres. The precast soffits are supported on steel angles on the sides of the top booms of the trusses and the span of the precast soffits is therefore about 10 ft. Between, and bearing on, the precast soffits are solid precast concrete slabs (*Fig.* 1), which are 2 in. thick, 9 in. wide, and 18 in. long, and act as permanent shuttering for a 4-in. cast-in-situ concrete slab. The cast-in-situ concrete is reinforced to carry 4-in. dwarf reinforced concrete walls which support the upper slab at 6-ft. 3-in. centres. Successive stages in the construction of

Fig. 1.—Laying Precast Slabs on Upper Part of Roof.

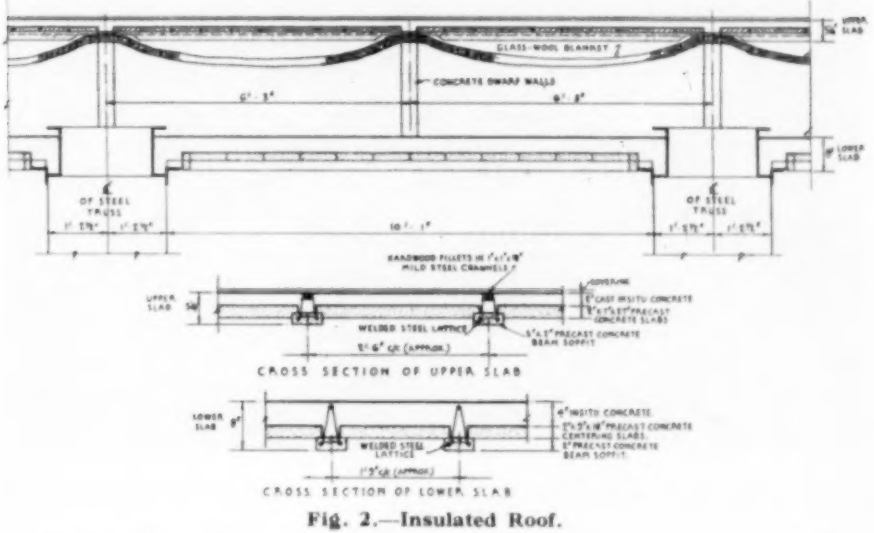

Fig. 2.—Insulated Roof.

the lower slab and dwarf walls are shown in *Fig.* 4. The glass-wool blanket is shown in position in *Fig.* 3, which also shows the construction of the upper slab in progress.

The upper slab is also a composite slab, but it is only 5¼ in. thick and is designed to carry a superimposed load of 30 lb. per square foot. It is constructed with precast beam soffits and precast slabs similar to those in the lower slab, but the precast beam soffits are at 2-ft. 6-in. centres and the precast slabs are 2 in. thick, 7 in. wide, and 2 ft. 3 in. long. The ends of the beam soffits are notched as shown in *Fig.* 3 to permit the glass-wool blanket to pass over the supports. The cast-in-situ concrete in the upper slab is 2 in. thick.

The steel lattice embedded in the precast beam soffits in the lower slab comprises two bottom bars (in the precast part) and one upper bar (in the cast-in-situ concrete) connected by diagonals welded to the bars. The bottom bars provide the main reinforcement for the slab and the top bar is the compression flange while the lattice is acting temporarily as a beam supporting the precast slabs and the wet cast-in-situ concrete. The lattice in the precast beam soffits, which span about 6 ft., in the upper slab is similar except that instead of a top bar there is

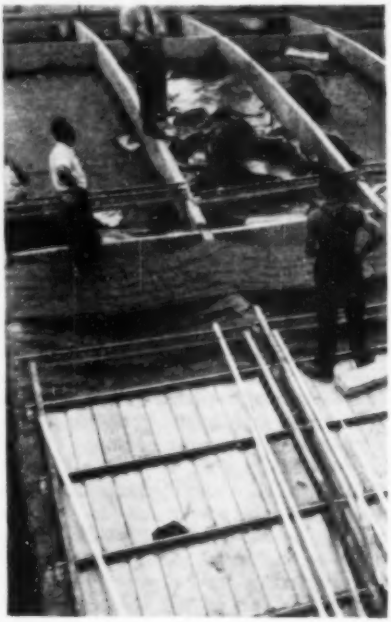

Fig. 3.—Insulating Blankets between Lower and Upper Parts of Roof.

1-in. by 1-in. by ⅛-in. mild steel channel in which a hardwood strip is placed for

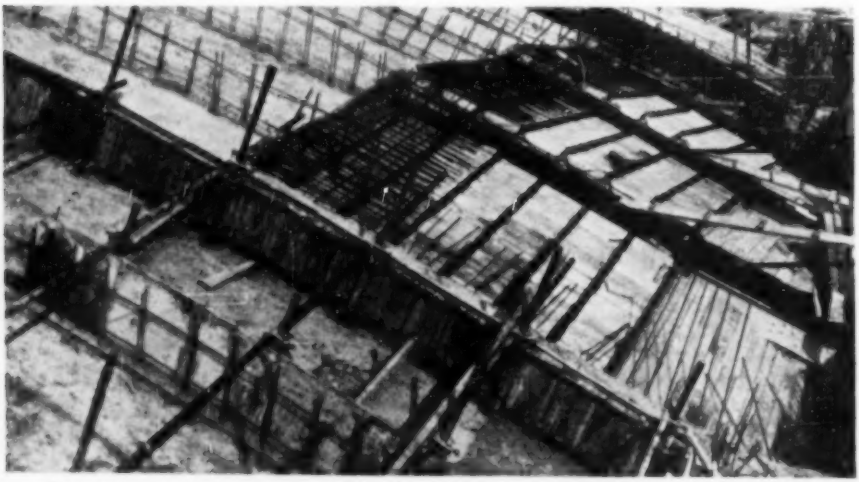

Fig. 4.—Lower Part of Roof under Construction, showing Dwarf Walls to carry Upper Part.

fixing the hardboard and copper sheet. The weight of each of the longest precast beam soffits is about 145 lb. and the precast slabs weigh about 28 lb. each. The precast members were lifted to the roof (a height of about 115 ft.) on a platform hoist. The beam soffits were placed on their supports by two men working from beams and slabs already laid.

The architect for the hall is Mr. Robert H. Matthew, A.R.I.B.A., chief architect of the London County Council, directing a team of architects. The consulting civil engineers are Messrs. Scott & Wilson. The main contractors are Messrs. Holland & Hannen and Cubitts, Ltd. The subcontractors for the precast roof members are Triad Floors, Ltd.

Measurement of Consistency.

A DEVICE, which is attached to a concrete mixer and is used by the Corps of Engineers at Allatoona Dam, U.S.A., measures the consistency of concrete while it is being mixed and therefore enables changes of consistency to be detected immediately. The device (*Fig.* 1) is a small U-shape bucket placed inside the drum and mounted on a shaft that extends through the wall of the drum. An arm on the end of the shaft makes electrical contacts when the shaft turns. Pressure of the concrete being mixed tilts the bucket, causing the shaft to rotate and the arm to turn through a distance depending on the stiffness (or workability) of the concrete. The amount of turn is recorded electrically. Comparison with a predetermined amount of turn shows whether the workability of the concrete being mixed is the same as that required.

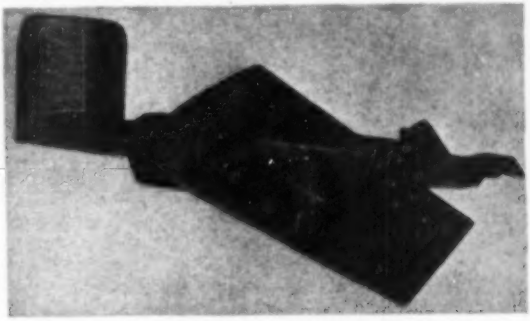

Fig. 1.—Consistency Recorder.

The device is suitable for any size of aggregate up to 6 in., but, according to a report in " Engineering News-Record ", the recordings may fluctuate because of a loose stone hitting the bucket; and it is said not to operate well in rain or damp weather.

Seventy-five Years of Reinforced Concrete.

A NOTE in " Beton- und Stahlbetonbau " records the seventy-fifth anniversary of the foundation of Wayss & Freytag A.G., the well-known German firm of engineers and contractors. The firm's interest in reinforced concrete started in 1884, when it acquired the rights in Germany of the Monier system. In the year 1901 Dr. Emil Mörsch, who has for long been recognised as a leading authority on reinforced concrete, joined the firm and has been responsible for its technical developments and research work in reinforced concrete; at the age of 80 years, Dr. Mörsch has had nearly fifty years' service with the firm. In the year 1934 the firm acquired a licence to operate in Germany M. Freyssinet's system of prestressed concrete, and it has been concerned with many other developments in concrete.

Prestressed Concrete Structure at an Exhibition.

THE "sword" illustrated in *Figs.* 1 and 2 is a prestressed concrete structure erected at a building exhibition held recently by the Ministry of Works at Leeds. The top of the shaft was 51 ft. above the ground, and the foundation extended 4 ft. below the ground. The topmost 6 ft. was of transparent material enclosing a light. The concrete shaft, which can be dismantled for erection elsewhere, is 48 ft. long, the cross section being rectangular, 2 ft. by 1 ft., for a distance of 13 ft. from the bottom, tapering to a diamond-shape, 8 in. by 4½ in., at the top. The shaft is prestressed by one ¾-in. bar (A) and two ¾-in. bars (B). The "wings," which extend 17 ft. on either side of the shaft, are 2 ft. wide, 3 in. thick at the tips, and 6 in. thick at the shaft, and are prestressed by three ¾-in. bars (C) extending from tip to tip. The bars are of special alloy steel, and the prestressing system is the Lee-McCall system which was described in this journal for April, 1950. Some mild steel binders are provided in the concrete immediately below the anchor-plates against which bear the nuts threaded on the ends of the bars. The total weight of the bars is about 5 cwt.

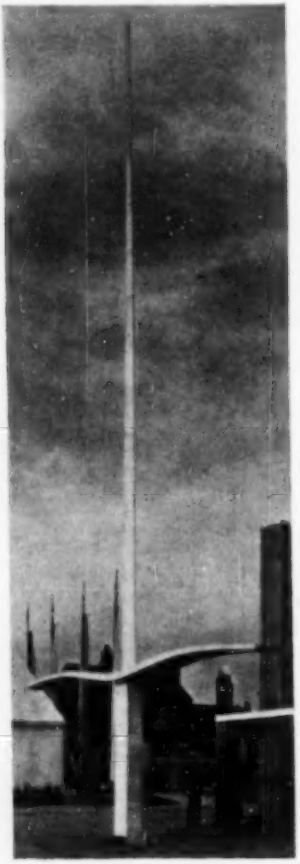

Fig. 1.

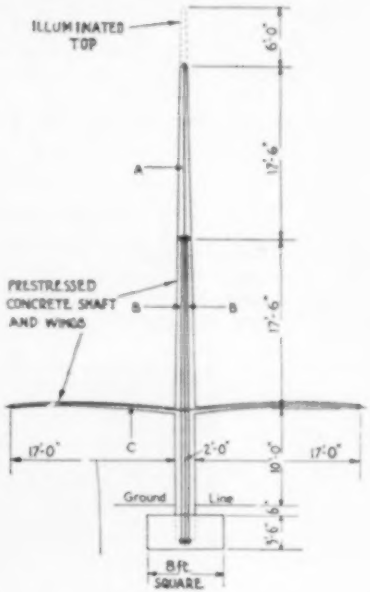

Fig. 2.

The concrete has a crushing strength of not less than 8000 lb. per square inch. The members were cast in short sections, assembled with dry joints, and prestressed on the site. The structure, which was designed by Mr. Donovan H. Lee, M.Inst.C.E., weighs about 5 tons and was made and erected by a mobile crane by Messrs. William Nicholson & Son (Leeds), Ltd.

Precast Concrete in Government Offices.*

THE plan of single-story temporary offices erected by the Ministry of Works for various Government departments comprises a number of buildings each 180 ft. long and 36 ft. wide internally and connected at the centre of their sides by a corridor. Some of the buildings have a frame of precast concrete, but more recent structures have prestressed concrete roof beams.

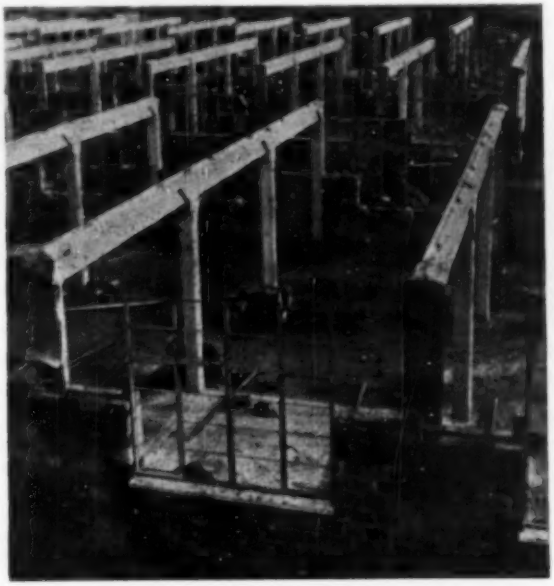

Fig. 1.—Precast Reinforced Concrete Frames.

Precast Reinforced Concrete Frames.

Precast reinforced concrete frames (*Fig.* 1) comprise columns, erected on a grid of 12 ft., supporting beams which carry reinforced concrete roof slabs. The roof slabs are covered with a screed laid to a slope, and bituminous felt. The buildings in *Figs.* 1 to 3 are at Hinchley Wood, Surrey, and embody precast concrete beams and columns, cast-in-situ hollow-block roof slabs, and concrete-brick cavity walls. The aggregate in the columns and beams is ½-in. crushed gravel and washed pit-sand. The concrete, which was vibrated, had a slump of less than 1 in. and was mixed in the proportions of 1 : 2 : 4, rapid-hardening Portland cement being used in beams cast in the winter and ordinary Portland cement in the summer. The columns are 9 in. by 7 in. by 11 ft. long, a splayed head carrying the beams. The wall columns have a rib projecting on two sides to receive the concrete bricks forming the inner leaf of the walls. The reinforcement in the columns is four ½-in. bars with ⅛-in. links at 6-in. centres. The beams are 12 ft. long, 9 in. wide at the top, and have a web 5 in. thick; the total depth is 14 in. There are two ¾-in. bars in the bottom and two ½-in. bars in the top, with ⅛-in. binding at 9-in. and 18-in. centres. Two links for attaching lifting-tackle project from the top of the beam. A bolt, built in at the time of casting, projects from the top of each column and,

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when the beam and column are erected, the bolt passes through a hole or slot formed at the end of the beam, thus securing the two members. A recess is formed for a large square steel washer which fits on the end of the bolt.

Figs. 2 and 3 show the erection of a beam and a column. In-situ concrete foundations are constructed with recesses to receive the columns. Timber shuttering is used for some of the bases for the wall columns, but the bases for the inner

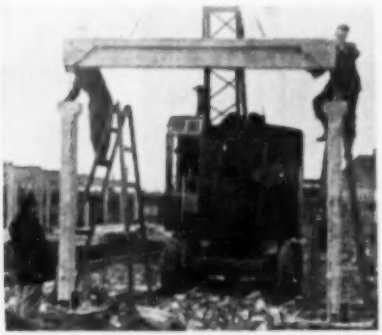

Fig. 2.—Erecting Precast Beams.

columns are cast within permanent shuttering of concrete blocks. One column is shown in *Fig.* 3 slung from a crane and being lowered into position and another column is shown wedged in the recess in the foundation. *Fig.* 2 shows a beam, also slung from a crane, being lowered on to the bolts projecting from the heads of

Fig. 3.—Erecting Precast Columns.

the columns. Each column weighs about 6 cwt. and each beam about 10 cwt.

Prestressed Concrete.

Similar buildings designed by the Ministry of Works comprise prestressed concrete transverse beams at 12-ft. centres spanning the 36-ft. width of the building, thereby enabling interior columns to be omitted. As it is not known in advance what type of roof covering will be used

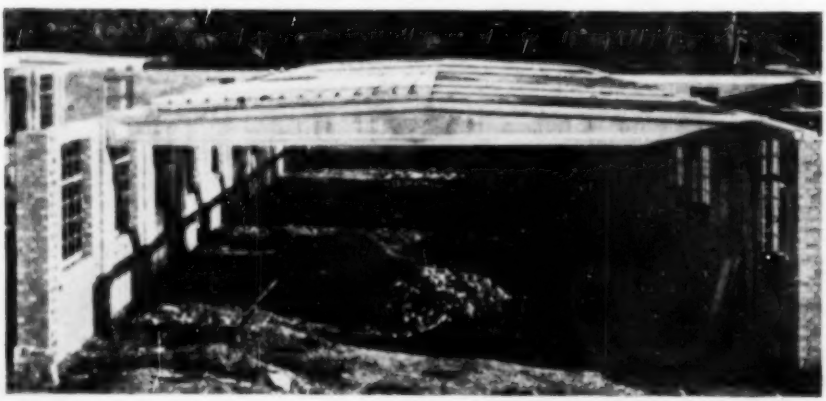

Fig. 4.—Prestressed Concrete Beams in Position.

the beams are designed for the heaviest probable load of 900 lb. per linear foot. The slope of the roof is obtained by sloping the tops of the beams at 1 in 20. Eighty beams of this type are used in a building at Reading (*Fig.* 4). The beams (*Fig.* 5) are prestressed by 178 No. 14-gauge wires embedded in the concrete. The depth at midspan is 27 in. and at the ends 18 in. The width of the flanges is 9 in., and the overall length is 37 ft. 4 in. The wires are stretched to an initial stress of 90 tons per square inch before the concrete is placed. There is a space of 1 in. between each vertical row of wires and ¼ in. between each horizontal row. The mixture of the concrete is 1 : 1·5 : 2·45 and the water-cement ratio 0·35. The compressive strength of the concrete is about 6000 lb. per square inch at seven days and 8000 lb. per square inch at 28 days. A small quantity of mild steel reinforcement is provided in the form of wire helices wrapped around the stretched wires at each end of the beam. The lifting hooks are the only other steel incorporated. The total weight of wire in each beam is about 1 cwt. Advantage was taken of the dead load of the beam in the design, and the concrete stresses in the bottom flange, when the beam is not loaded, are 2400 lb. per square inch at midspan and 2600 lb. per square inch near the ends. Under the working load there is generally a small compressive stress in the bottom flange except at a point 5 ft. from the middle of the span,

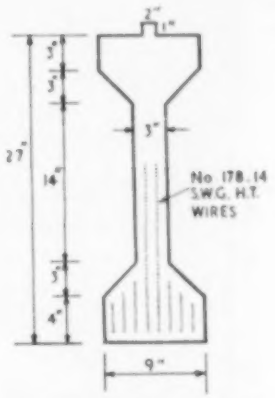

Fig. 5.—Cross Section of Prestressed Concrete Beam.

where a small tensile stress may occur. In these prestressed concrete beams having straight stretched wires and a sloping-top flange, the critical sections are at about one-eighth of the span on both sides of the centre of the span.

Before the beams were erected two were tested by being laid on their side with the top flanges facing each other. Two jacks were inserted between them at points 10 ft. from the ends, which were strapped together. Hair-cracks were observed when the total force acting on the beams was 23 tons, the applied bending moment being 3,000,000 in.-lb. corresponding to a stress at the bottom edge, at the point

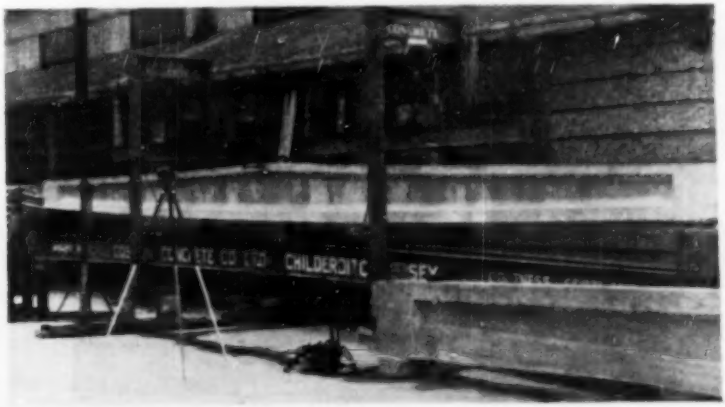

Fig. 5.—Testing a Prestressed Concrete Beam.

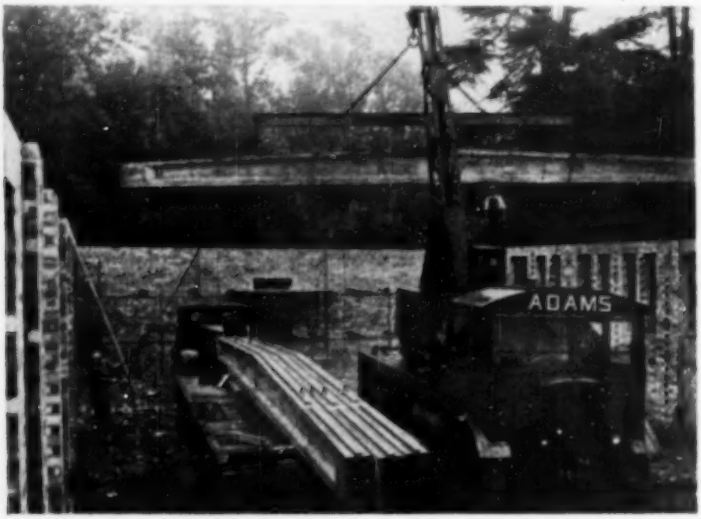

Fig. 7.—Erecting Prestressed Concrete Beams.

where the cracks were first observed, of 4170 lb. per square inch. The precompression at this point was calculated to be between 2700 lb. and 2400 lb. per square inch at the time of the test. The tensile stress in the concrete would therefore be about 1500 lb. per square inch, which seems to be excessive. If it is assumed that cracking actually occurred when the load-deflection curve first departed from a straight line, the tensile stress would be between 900 lb. and 1200 lb. per square inch, which agrees with the stresses observed in other tests. A test of a similar beam 24 in. deep at midspan is shown in progress in *Fig.* 6.

The beams in the structure in *Fig.* 4 were cast several in one line in a factory, and were transported to the site four at a time on low-load lorries. They were

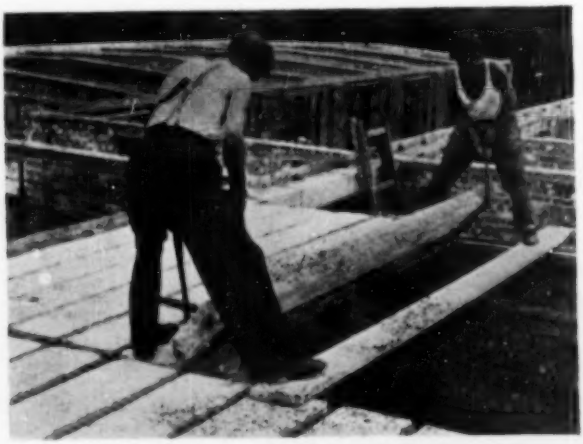

Fig. 8.—Placing Prestressed Concrete Roof Slabs.

erected by being lifted at two points (*Fig.* 7) and placed on mortar beds on padstones on brick piers. For ease of erection, one end wall of the building was omitted and erection proceeded from the other end. The beams support hollow prestressed concrete roof slabs (*Fig.* 8) 4½ in. deep and weighing 30 lb. per square foot. The roof is covered with a screed and felt for the purpose of weather tightness and insulation.

The cost of the structure is about the same as that of a reinforced concrete structure, but the large space uninterrupted by columns is an advantage.

The beams and columns in *Figs.* 1 to 3 were made by Messrs. F. Bradford & Co., Ltd., and were erected by direct labour by the Ministry of Works. The beams in *Figs.* 4 and 7 were made by Concrete Development Co., Ltd., and the beam in *Fig.* 6 was made and tested by Costain Concrete, Ltd. The hollow roof slabs in *Fig.* 8 were made by Concrete, Ltd.

The New Motor Road from Paris to the West.

THE new motor road westward from Paris comprises a main road about six miles long from St. Cloud to Rocquencourt where two branches bifurcate, the northern branch being about eight miles long and the southern five miles. At St. Cloud there is a boulevard 132 ft. wide terminating at a tunnel 2760 ft. long, 56 ft. wide at the springings, and of horseshoe cross section. From the tunnel to Rocquencourt there are two carriageways each 29 ft. 6 in. wide separated by a strip 8 ft. 3 in. wide. The branches each comprise two carriageways each 23 ft. wide separated by a strip 7 ft. 4 in. wide.

The work was commenced before the last war, at the outbreak of which the tunnel, bridges, and earthworks of the main road and the northern branch were almost complete, and work was proceeding, though less advanced, on the southern branch. The bridges and the junction at Rocquencourt were described and illustrated in this journal for July, 1939. Work ceased in 1941, when the carriageways of the main road and the northern branch were partly completed. Work recommenced after the war and the tunnel was opened in 1946. The remainder of the main road was opened in January, 1950, and the southern branch in July, 1950.

According to "Travaux," the latest carriageways of the main road have a total thickness of 19½ in., the lower 2 in. of which is a drainage layer of clinker upon which is laid 4¾ in. of rolled hardcore covered by about 3 in. of river sand. The remaining thickness of about 9½ in. is a concrete slab, the lower 7½ in. of which is of concrete of the following proportions: 2976 lb. of ⅞-in. to 2¾-in. coarse aggregate, 1499 lb. of dry sand, 772 lb. of Portland cement, and 26½ gallons of water. The proportions of the concrete in the 2-in. wearing surface are 3086 lb. of broken prophyry graded from ⅞ in. to 1¼ in., 1349 lb. of dry sand, 772 lb. of Portland cement, and 27½ gallons of water. The carriageway is laid to a transverse fall of 1½ in 100. A layer of waterproofed paper is placed below the concrete.

The carriageways of the southern branch comprise 2 in. of clinker, 4 in. to 4¾ in. of sand, and 9½ in. of concrete, the proportions of which are 2780 lb. of ⅞-in. to 2¾-in. aggregate, 1642 lb. of dry sand, 772 lb. of Portland cement, and 27½ gallons of water. No special wearing surface is provided.

The concrete slabs are laid in panels 26 ft. long with expansion joints between every group of three panels, and two dummy-joints intermediately. Reinforcement is provided where the subsoil is weak.

Prestressed Concrete Sewer Tunnels near Paris.

THE construction of 1½ miles of prestressed concrete sewer in a tunnel (*Fig.* 1) was recently completed at two points in the western suburbs of Paris. At La Frette the tunnel is 585 ft. long and 13 ft. 6 in. diameter, and forms a siphon under the river Seine. At Montesson a tunnel 7000 ft. long and 15 ft. 6 in. diameter is driven through loose sand and sandstone below the water table in a loop of the river. These two prestressed concrete tunnels are joined and continued by an ordinary concrete-lined tunnel through impermeable strata.

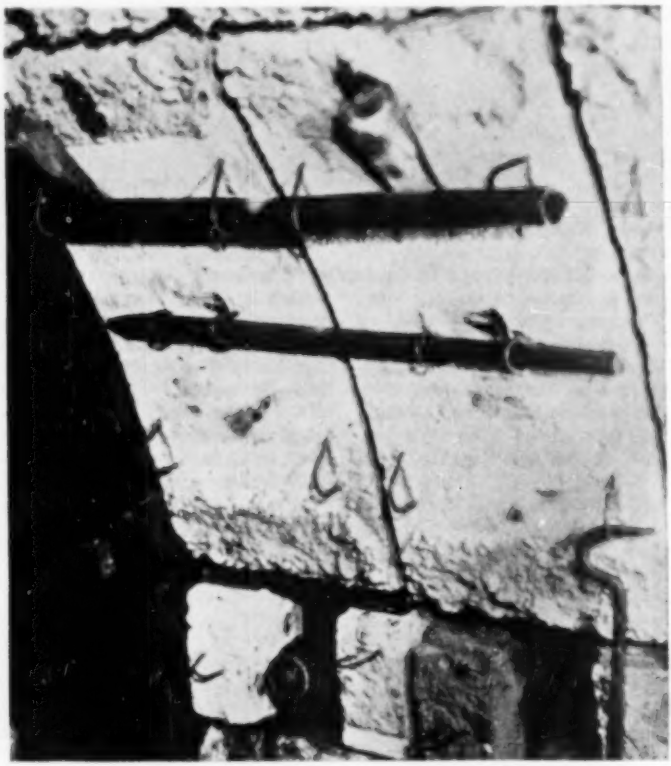

Fig. 1.—Voussoirs in Position.

The cross section of the prestressed concrete tunnel in *Fig.* 3 shows, on the left-hand side, the construction before prestressing and the completed tunnel on the right. The voussoirs (V) forming the lining were precast in steel moulds on vibrating tables in a factory some miles from the site. The concrete contains 500 lb. of rapid-hardening Portland cement per cubic yard. An ordinary voussoir is 4 ft. 8 in. by 1 ft. 9 in. by 1 ft. 4 in. thick, weighs 1650 lb., and is accurate in size to 1 mm. In each voussoir there is a hole (H) for the injection of mortar (M), and on the outer face there is a shoulder against which bears a tensioning hoop (P). The inner face is roughened to provide a key for rendering.

The voussoirs were placed in position in rings behind a driving-shield (S) compris-

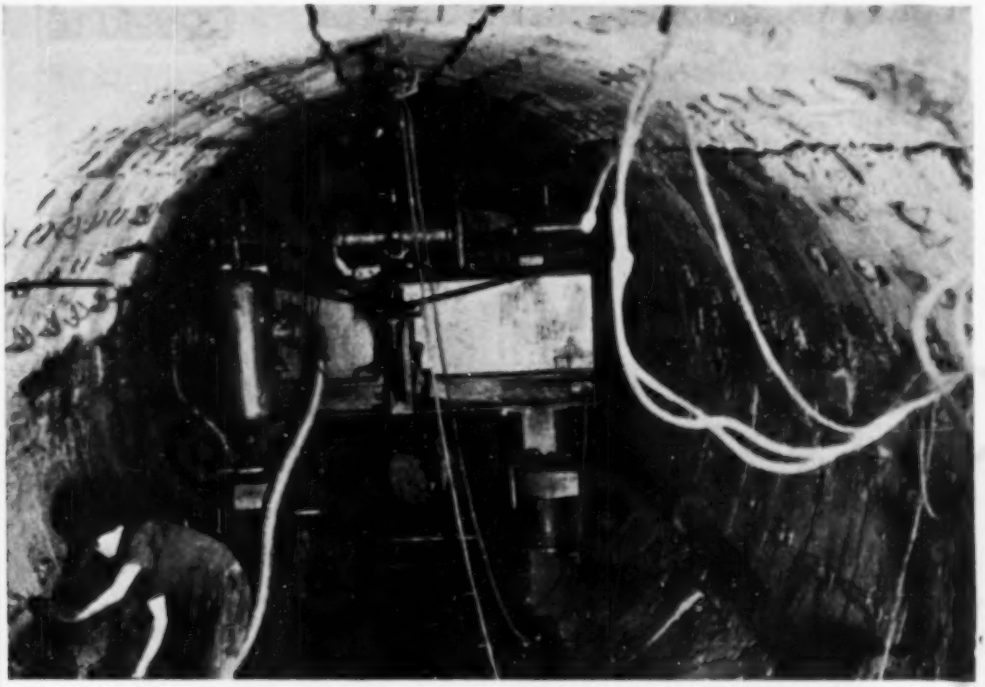

Fig. 2.—Tunnel Lining before Rendering, showing Erecting Machine and the Pump for the Moving Shield.

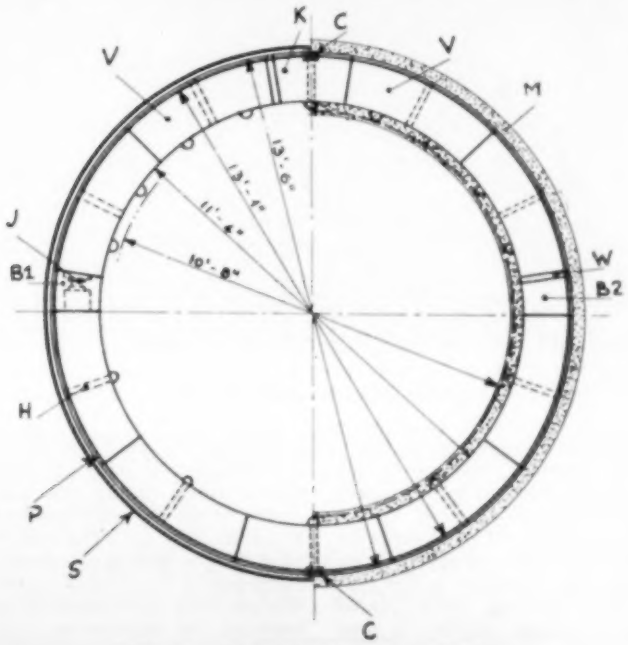

Fig. 3.—Cross Section through Sewer.

ing a cylindrical steel-plate shell the rear end of which overlapped the last completed ring of voussoirs. The forward end carried the cutting edge. The shield was moved forward by fifteen jacks bearing against the last completed ring of voussoirs, thus effecting a longitudinal

open. In *Fig.* 5 it is seen putting a key voussoir (K in *Fig.* 3) in position.

A hoop of 0·2-in. stainless steel 10 in. wide and in two parts connected by clips (C) was then placed around the newly-placed ring of voussoirs and two 50-ton jacks (J) were placed between the upper

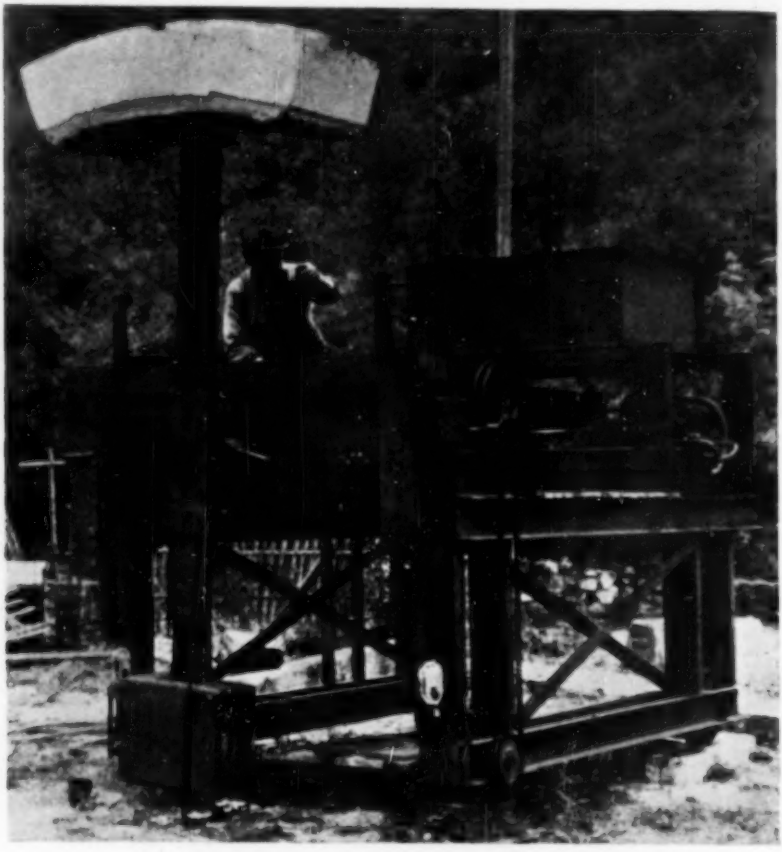

Fig. 4.—The Erecting Machine.

compression of the tunnel lining. When the shield had advanced sufficiently, the next ring of voussoirs was placed in position by an erecting machine (*Fig.* 2), the upper voussoirs being held in position by dowel-bars passing through steel loops projecting from the voussoirs in the preceding ring and the ring being placed (*Fig.* 1). The erecting machine is seen more clearly in *Fig.* 4 where it is in the

and lower halves of the ring. Spaces for the reception of the jacks (seen in *Fig.* 1) were formed during the placing of the voussoirs by the insertion of blocks (B1) and (B2) (*Fig.* 3). The halves of the ring were then jacked apart until the required extension of the steel hoop was obtained. Wedges (W) were inserted in the space between the semicircles and the jacks removed. The circumferential compres-

Fig. 5.—Placing a Key Voussoir.

sive stress thus obtained did not exceed 420 lb. per square inch as it was additional to the stresses induced by the pressure of the surrounding earth.

The process of moving forward the shield, placing the voussoirs, and prestressing the ring took seven hours. As three shifts were worked, three rings were placed in each 24 hours representing a forward movement of 5 ft. 6 in. The space between the bore as excavated and the concrete lining was then filled with cement-mortar injected through the holes in the voussoirs. This operation was the work of a second gang. A third gang followed with a machine for rendering the internal face of the lining. Construction of the main length of the tunnel at Montesson was carried out in opposite directions from a central shaft. The work was done in compressed air at a pressure of 30 lb. per square inch.

The precompression of the rings ensured a watertight tunnel through the permeable strata. The minimum amount of ground between the top of the tunnel and the bed of the river is 33 ft., at which point the river is 26 ft. deep but may be up to 42 ft. deep when in flood.

The tunnels were designed by Société Technique pour l'Utilisation de la Précontrainte (Procédés Freyssinet).

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The Design of Cylindrical "Shell" Roofs.*

THOSE of our readers who are interested in thin reinforced concrete vaults and domes, commonly known as "shell" roofs, will welcome a book* written by a Danish engineer and printed in the English language. In our opinion this is the most useful work on the subject so far published. Vol. I, which comprises 360 pages and is well illustrated, describes the various methods of designing cylindrical shells, and particularly methods that need only an elementary knowledge of the theory of structures. It is claimed that by the methods recommended all the calculations can be made by slide-rule. The worked examples are particularly useful. The method of designing a new type of roof, called a "rigid vault", is also explained. The following summary of the author's notes on the analysis of "shells" are given by permission.

A proper "shell" structure has a curved surface, the thickness of the slab forming the surface being small compared with its area. Because of its negligible resistance to bending (except near edges), the slab must be supported, or stiffened, in such a way that the shell is so rigid that loads produce little bending but comparatively large direct stresses. Rigidity of a cylindrical shell is obtained by transverse webs at the supports. If there are only two such stiffeners, the membrane stresses are statically determinate even though the shell cantilevers beyond the stiffeners. If the shell is continuous over three or more stiffeners it is statically indeterminate. A reinforced concrete shell of uniform thickness is considered to be homogeneous. A shell is isotropic if the elastic properties at any point are the same in all directions. Therefore a shell of homogeneous material but of uniformly-varying thickness is isotropic, but if the shell is stiffened by ribs between the supports, it is anisotropic. An anisotropic shell is therefore non-homogeneous, and strictly should be considered as such in the stress calculations, but in practice this complexity is unnecessary if the longitudinal deformation is such that several ribs are covered by each semi-wave of deformation.

Sometimes the bending stresses are so small that the state of stress can be determined in accordance with the membrane theory (see this journal for April, 1948), the bending being neglected. In many shells the bending stresses at the edges cannot be neglected, but the membrane theory can be used for parts remote from the edges. In the latter case there are stresses due to discontinuity at the edge of the shell (called by the author "perturbational stresses"), and in shells of double curvature and some cylindrical shells it is sufficient in the central part of the shell to consider the undisturbed membrane stresses. For other types of shells the stresses may be essentially different from the stresses calculated in this manner, and it is often convenient in these cases to calculate the stresses as the sum of the membrane stresses and the disturbances originating from the edges.

Two conditions only have to be fulfilled at each end of an ordinary beam or at the edge of a loaded plate, the deflection of the beam being expressed by an equation of the fourth degree to which the partial differential equation of the plate is analogous. In shells, for which the differential equation is of the eighth degree, four conditions must be satisfied at each edge point, the eight quantities to be considered being three displacements and three related forces in a three-dimensional co-ordinate system, one angular displacement, and one related bending moment. The simplest edge conditions occur at a free edge where the three forces and the bending moment are each equal to zero, and at a fixed edge where the four deformations are each equal to zero. For elastically-supported or partially restrained edges, all eight factors must be considered with consequent complications in the calculation.

It is, however, possible to design most cylindrical shells by new methods that require no more mathematical skill than is required for statically-indeterminate structures, and sufficient accuracy is obtained by the use of a slide-rule. Well-known elementary methods are in some

* "Cylindrical Shells". Volume I. Cylindrical Roofs. By H. Lundgren. (Copenhagen: The Danish Technical Press. 1949. In the English language; numerical examples in the metric system. Obtainable from Books and Careers, Ltd., 84 Victoria Street, London, S.W.1. Price 28s.; by post 28s. 10d.)

cases, at least, suitable for preliminary designs and in many cases for final designs.

The Beam-Method.

One approximate method is the beam-method, which is applicable to long shells. In this method, the stresses in the shell and edge-beams are analysed exactly as an ordinary homogeneous simply-supported beam, the moment of inertia of which is calculated simply from formulæ which make use of the first two terms of a power series. (If the formulæ are not available, a semi-graphical method can be used.) The application of this method to symmetrically-loaded symmetrical shells, subjected to bending in a vertical plane, is common, but it can be extended to unsymmetrical structures, such as saw-tooth roofs, in which bending may be in any plane. In the latter case the position of the centroid is determined and the stress analysis is made preferably by semi-graphical means for bending vertically and horizontally. For a purely arithmetical analysis it is advantageous to use a system of co-ordinates other than the principal axes, and formulæ developed for this case facilitate the calculation. Long shells subjected to torsion can be analysed by a theory, the principal of which is not new, analogous to the theory of bending.

In the extended beam-method the deformations due to the transverse bending moments are also considered, so that structures such as those comprising a number of interconnected parallel long shells may be analysed by an elementary calculation.

Effects of Disturbance.

The stresses in short shells arising from disturbances due to discontinuity of the load and to the effect of the edges cannot be considered by elementary calculations; it is necessary to apply the general analytical method, which is made easier since the shell may be considered as a circular arc of radius equal to the radius of curvature at the point of disturbance. The method of applying formulæ derived on this basis to the design of a short shell is first to calculate the stresses in the shell by the membrane theory, according to which the membrane shearing forces cause tension in the edge zone. To this tension must be added the tensile force due to edge disturbances. The reinforcement required to resist the shearing forces is determined, and the transverse restraint bending moment at the edge can then be calculated by substitution in a simple formula.

An important limitation is that the shell is assumed to be simply supported. Although short shells are commonly continuous over several spans, it is not practicable at present to apply the general analytical method to other conditions. Continuous short shells can, however, be approximately analysed as rigid vaults, the formulæ for which are based on the analytical method.

If the analytical method is applied to tubes, formulæ can be derived for the stresses due to disturbance in the shell near transverse stiffeners, distinction being made between the primary bending conditions and the secondary conditions, which are not satisfied in elementary analyses of shells.

Repetitive Method.

The new repetitive method (called by the author the "iterative" method) of analysing shells is applicable principally to shells of a length between once and twice the width. The basis of the method, which is a generalisation of the beam-method, is that the central part of a shell can be considered as an arch which, through the remainder of the shell, is elastically supported on the stiffeners. By means of a number of repeated calculations (in practice two or three), the computed stresses approach the exact values. In the first calculation the shape of the cross section is assumed to remain unchanged, that is infinite flexural rigidity of the shell is assumed, and this is identical with the calculation by the beam-method. From the transverse bending moments the deformations of the cross section considered as an arch can be computed, and it can be shown that these deformations result in a redistribution of the axial normal forces. The subsequent calculations are similar to the first. In the calculations secondary factors such as torsional moments in the shell, shearing strain, etc., are considered, which it is not possible to do in the beam-method. Compared with the analytical method, which is purely mathematical, the repetitive method has the advantage of giving a clear physical meaning to each step of the calculation.

The method requires the same amount of work for a shell of any cross-sectional shape and any type of load, but requires more calculation than the beam-method. It makes clear the relation between the membrane theory and the beam-method, giving at the same time criteria for the application of the latter method. Like the analytical method, it is restricted to simply-supported shells.

An important aspect of the repetitive method is the rate of convergence and, for symmetrical cross sections at least, practical formulæ can be derived for a circular shell without edge beams, and corrections applied for the edge beams, varying curvature, and varying rigidity.

Analytical Method.

The general analytical method of designing isotropic cylindrical shells (without ribs) is based on the differential equation of the eighth degree, but a few modifications enable a practicable procedure to be evolved. Some simplifications, such as the omission of all secondary factors, may lead to unreliable results. Tables of data derived from accurate formulæ are, however, a considerable aid in applying the analytical method to shells of any ratio of length to width, and of any radius and thickness. Tests show that short reinforced concrete cylindrical shells behave as assumed in the analytical method if the concrete does not crack, but thereafter the deflections increase more rapidly at points of disturbances.

Exact stress-displacement relations can be derived by the analytical method for anisotropic cylindrical shells (with ribs) without neglecting the lateral contraction. By introducing the greatest permissible number of simplifications, formulæ can be derived to determine the stresses produced by a concentrated load on a shell with any number of ribs.

Theory of Rupture.

A new method of analysis is by the theory of rupture, which has yet to be confirmed by tests. In this theory, applied to long reinforced concrete shells, the axial tensile stresses are assumed to be resisted by the reinforcement in which ordinary tensile stresses are allowed. The compressive stress in the concrete is assumed to be equal to the permissible stress in direct compression, and is assumed to be uniformly distributed within the compressive zones, the extent of which is determined by analogy to an ordinary reinforced concrete beam. The zones are so defined that the total longitudinal reinforcement is a minimum. To simplify preliminary calculations each tensile or compressive zone is assumed to be concentrated at a single generatrix, called a "stringer". The approximate distribution of stresses in a shell is easily determined by the stringer-theory. When the shell has been calculated by the stringer-theory, which is so simple that it gives a general treatment of problems, a small correction only is required to revert to the theory of rupture. In statically-indeterminate cross sections, the redundant reactions at the edge generatrices are determined by considering the transverse moments in a similar manner to the theory of plasticity for continuous beams.

In the theory of rupture for short reinforced concrete shells, the calculation is divided into a calculation by the membrane theory and a calculation of disturbances, which involve extensional forces and bending moments. The theory is based on the principle that the carrying-capacity of the shell is not exhausted until both the extensional strength and bending strength are fully utilized. Hence, if additional longitudinal reinforcement is provided, the bending moments are reduced, and vice versa. Longitudinal stresses and statically-indeterminate sections are considered in the same way as in long shells. The stringer-theory can be developed from the assumption that the shell can resist transverse bending moments only. Bending moments in other directions can be taken into consideration, and it can be shown that the disturbed load may be divided into two parts, one of which produces only bending moments. Thus the shell can be considered as a combination of a membrane and a plate. The calculation of a plate is made by a theory of the lines of rupture (see this journal for August, 1950, page 279).

Selection of Method.

Although in general the beam-method applies to long shells, the membrane theory to parts of short shells, and the repetitive method to shells of intermediate length, it is difficult to decide when, for example, a shell is sufficiently

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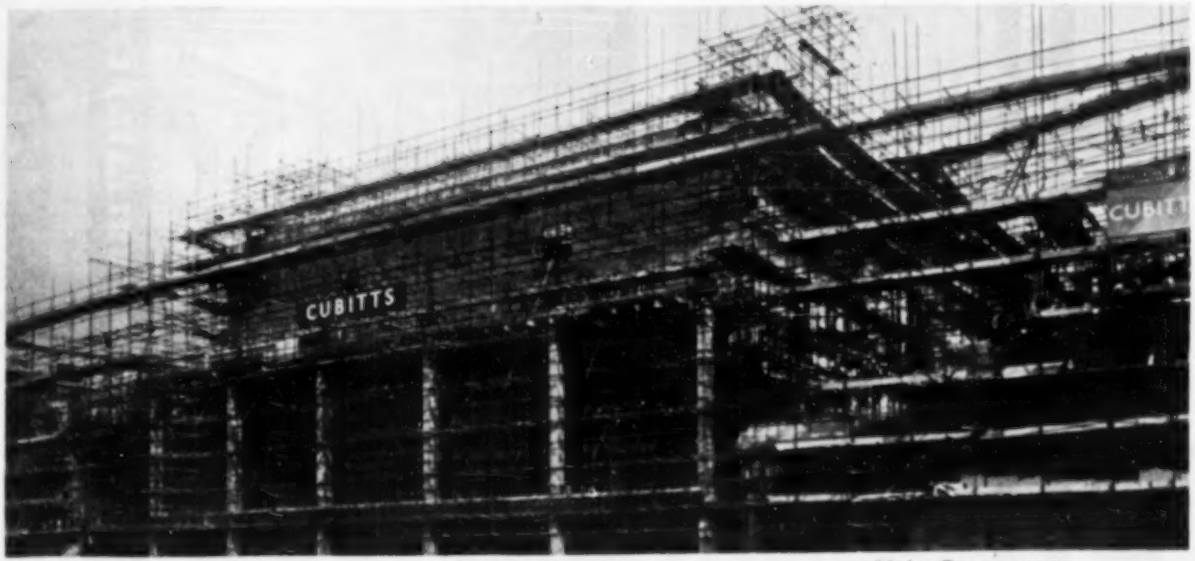

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long to allow the use of the beam-method. In doubtful cases the beam-method should be used first and, if the bending moments in the shell are small, the method is satisfactory. In shells with deep edge-beams, large bending moments are found when the beam-method is applied to the entire section, and in such cases it may be advantageous to consider the shell and edge beams separately. The applicability of the beam-method depends, however, on conditions other than the ratio of length to width. If a shell is stiffened by transverse ribs the rigidity is so great that the errors in the beam-method may be negligible even in a short shell. The repetitive method can be applied to test the applicability of the beam-method. If the rate of convergence is greater than unity, the beam-method may be applied for most common loads. If the load causes large bending moments the repetitive method may be used, although for isotropic cylindrical shells it may be easier to use the analytical method. Therefore the repetitive method is best chiefly for non-cylindrical isotropic shells of intermediate lengths, for which the analytical method is very complicated. Although the membrane theory gives a correct idea of the stress in those parts of short shells remote from the edges, investigation of the edge zones must be made by the analytical method, which is not an elementary problem.

Buckling.

The resistance of a shell to buckling must be investigated, and formulæ can be derived from the present knowledge of the buckling of tubes. A new empirical formula, which considers the special properties of reinforced concrete, gives results that are slightly conservative compared with the results of tests.

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Roads in Rural Areas.

Some alterations and additions to Memoranda Nos. 575 and 577 dealing with roads and bridges, issued by the Ministry of Transport, are given in Memorandum No. 653 (copies of which are obtainable free from the Ministry of Transport, Berkeley Square House, London, W.1). It is recommended that development plans should be based generally on a volume of traffic 75 per cent. in excess of the average daily traffic in 1938, but the possibility of new towns, new industries, etc., may justify a further increase. In the absence of exceptional local factors, an increase of 25 per cent. of the number of cyclists in 1938 should be assumed.

The limiting capacities of carriageways (expressed as the number of vehicles per day) are stated to be as follows: Two-line carriageways, up to 3000; three-line carriageways, 3000 to 6000; dual two-line carriageways, 6000 to 15,000; dual three-line carriageways, over 15,000. One-tenth of the daily traffic can be assumed to be the hourly traffic. If there are less than 600 vehicles per hour one cycle should be counted as a vehicle; when there are more than 600 vehicles an hour, two cycles should be counted as one vehicle. For roads leading to popular resorts, the assumed capacity should be reduced by one-third to allow for peak traffic. In general, central reservations should not exceed 10 ft. in width, but where the planting of trees or shrubs is intended the width may be increased to 15 ft. or 20 ft. The minimum width of the carriageway of trunk roads of Classes I and II should be 22 ft., but the width may be less where traffic is light and on Class III roads. There are some places where a single 30-ft. carriageway is suitable, as the limiting capacity of such a carriageway may exceed that recommended. There is not sufficient justification, because of traffic, to provide a distance of 70 ft. between the outside

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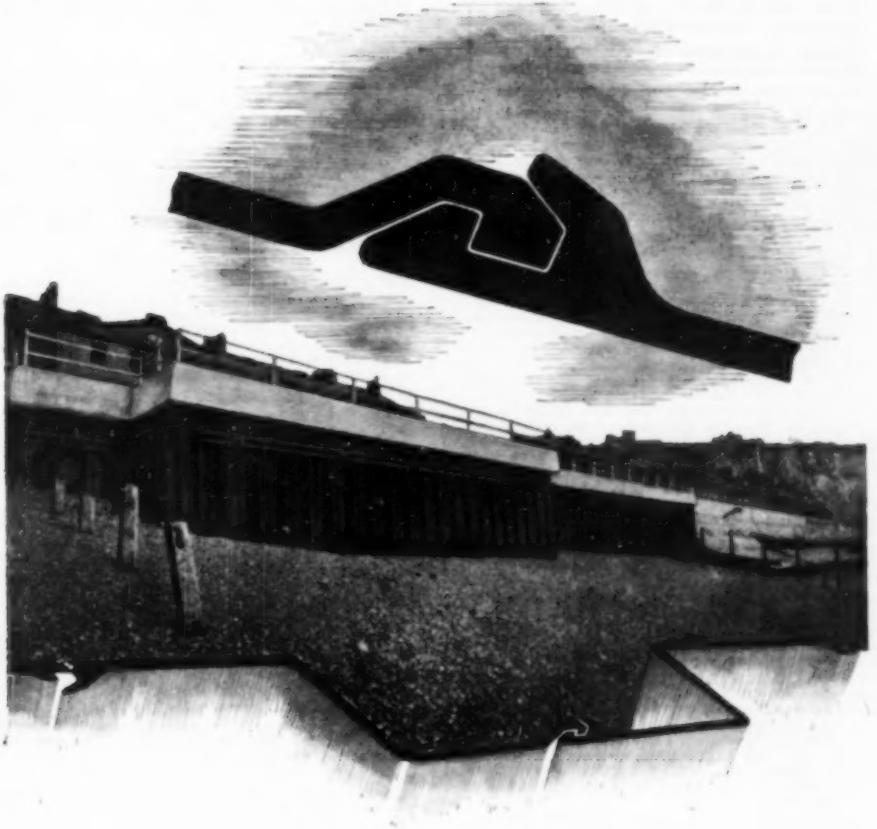

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F.H. 47

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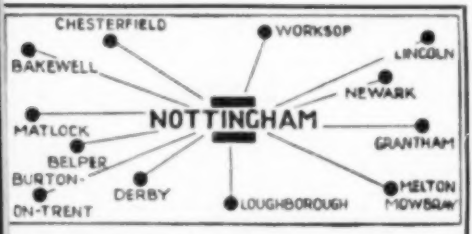
A gradient of 1 in 25 (instead of 1 in 30 as specified in Memorandum No. 575) should ordinarily be regarded as the maximum, but in hilly country a steeper slope may be necessary. Vertical visibility between points 3 ft. 9 in. above the carriageway and horizontal visibility on the centre-line of the carriageway depend on the speed that can be attained on the road; the distances for dual carriageways are as follows (the distances for single carriageways are given in brackets): 20 m.p.h., 100 ft. (150 ft.); 30 m.p.h., 150 ft. (250 ft.); 40 m.p.h., 250 ft. (450 ft.); 50 m.p.h., 350 ft. (650 ft.); 60 m.p.h., 500 ft. (900 ft.); 70 m.p.h., 650 ft. (1200 ft.).



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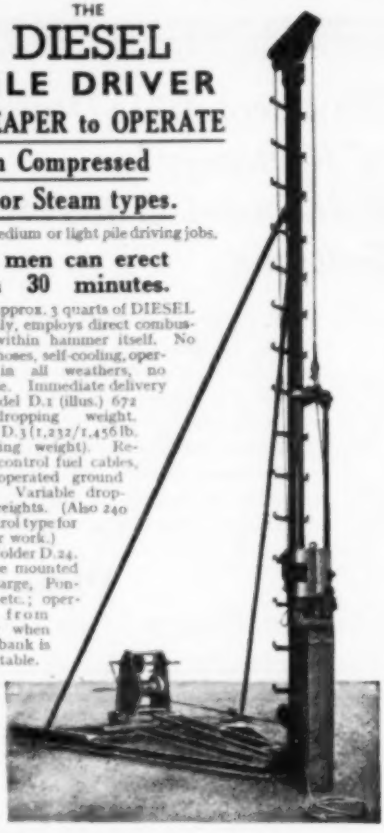

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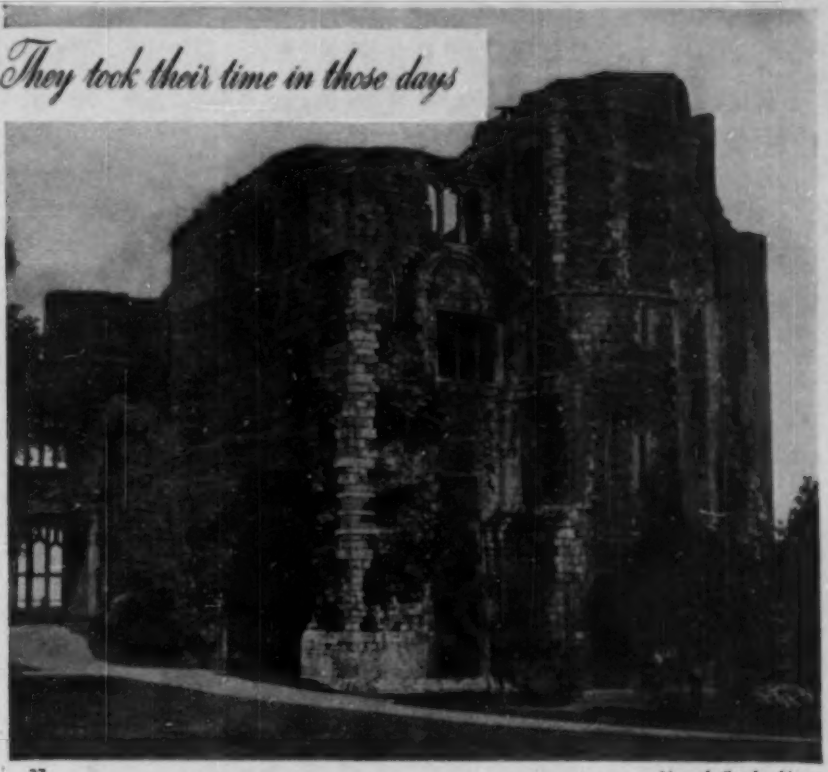

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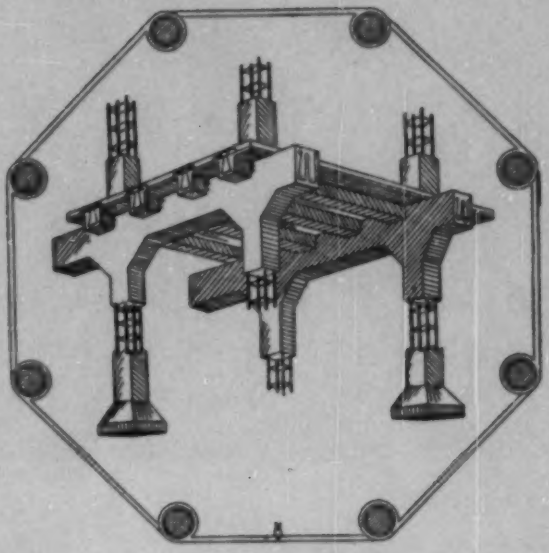

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